GREAT BRAK ISLAND – MEASURES FOR FLOOD DEFENCE

BY

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Declaration

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Abstract

This thesis firstly presents the analysis of large flood events, causing low-lying property in and around the Great Brak estuary to be inundated. The nature of several South African estuaries renders adjacent properties vulnerable to river and ocean related flooding. An important objective of the study is to develop a feasible form of flood defence for the Island, a residential development situated in the lower estuary basin, 250 m from the estuary mouth. The Island regularly experiences inundation due to high water levels in the estuary which have partially driven the need for the artificial manipulation of the estuary mouth berm using millions of litres of potable water.

The lowest property on the Island is situated at +2.2 m MSL whereas the highest flood level ever recorded in the estuary was +2.9 m MSL. This extreme water level was achieved by an equivalent 1 in 10-year flood, flowing into the estuary and coinciding with a closed estuary mouth. Research into climate change predictions for regional overland precipitation trends and for sea level rise have been included in the study. The sea level is predicted to rise by about 1 m in the year 2100 whereas the frequency of extreme floods is set to increase for the catchment area in the next century. Large storm events may cause direct wave attack on the shorefront properties of the Island due to the rising sea levels and the increase in storminess. Much higher extreme water levels are therefore expected in the estuary and a form of flood defence is therefore sought; also, to reduce the current need for “wasteful” water releases to artificially induce open mouth conditions

Various possible measures for flood defence have been identified for application either directly around the Island, in the surf-zone to dissipate wave energy or upstream of the estuary, at the Wolwedans Dam to attenuate large floods from rainfall in the catchment. A Multi-Criteria Analysis approach has been followed to objectively identify the preferred flood defence measure. The evaluation criteria were based on the hydraulic -, environmental-, and economic performance of the proposed flood defence measure. The Multi-Criteria Analysis identified a combined solution of an Armoured Dike- and Rock Revetment structure directly around the Island, to be the preferred solution.

The proposed solution was conceptually designed for various lifetimes and extreme flood conditions to find the least expensive option. An order of magnitude cost estimate for the construction of the proposed solutions was derived, which formed the basis of comparison to the costs foreseen if no flood defence measure is implemented (the Do-Nothing alternative). The study found that the proposed solution becomes economically feasible if designed for a lifetime of 33 year and more. The most attractive solution was found to be the combination of a +3.0 m MSL crest level Armoured Dike and a +5.0 m MSL crest level Rock Revetment, which is estimated to cost R95 million to construct.

Opsomming

Hierdie tesis bied eerstens ‘n analise van groot vloede aan, wat veroorsaak dat laagliggende eiendomme in en rondom die Groot Brak strandmeer oorstroom word. Die aard van verskeie Suid Afrikaanse strandmere, insluitend die Groot Brak strandmeer, maak aangrensende eiendomme kwesbaar vir rivier- en oseaanverwante oorstromings. ‘n Belangrike doel van die studie is om ‘n haalbare manier van vloedverdediging vir die Eiland te ontwikkel, wat ‘n residensiële ontwikkeling in die strandmeer is, ongeveer 250 m van die see af. Die Eiland beleef dikwels oorstroming as gevolg van hoë watervlakke in die strandmeer, wat gedeeltelik die behoefte aan kunsmatige manipulasie van die mond dryf, deur middel van miljoene liter drinkbare water uit die Wolwedans Dam te gebruik.

Die laagste eiendom op die eiland is geleë +2.2 m bo seevlak terwyl die hoogste vloedvlak ooit bereik in die riviermond aangeteken is as +2.9 m bo seevlak. Hierdie ekstreme watervlak was veroorsaak deur ‘n ekwivalente 1 in 10 – jaar vloed wat saam met ‘n toe mondkondisie gepaart gegaan het. Navorsing oor streeks voorspellings vir klimaatsverandering vir oorlandse neerslagneigings en vir seevlakstyging is by die studie ingesluit. Daar word voorspel dat die seevlak met ongeveer 1 m sal styg teen die jaar 2100 en dat die voorkoms van uiterste vloede in die opvangsgebied sal toeneem. Groot toekomstige storms sal veroorsaak dat golwe die Eiland kan bereik as gevolg van stygende seevlakke en die toename in stormagtigheid. Daar word dus baie hoër watervlakke in die strandmeer verwag en ‘n metode van vloedbeskerming word dus benodig; ook om die huidige verkwistende water loslatings, om die mond kunsmatig oop te spoel, te verminder.

Verskeie moontlike maatreëls vir vloedverdediging is geïdentifiseer vir aanwending direk rondom die Eiland, in die brander sone om golfenergie te demp en stroomop van die riviermond, by die Wolwedansdam om groot vloede te demp wat veroorsaak word deur reënval. ‘n Multi-Kriteria-Analise benadering is gevolg om die beste vloedverdediging maatreël objektief te identifiseer. Die evalueringskriteria is gebaseer op die hidrouliese-, omgewings- en ekonomiese prestasie van die voorgestelde vloedverdedigingsmaatreël. Die Multi-Kriteria-Analise het ‘n gekombineerde oplossing van ‘n bewapende dyk en ‘n ruklipkeermeer vir aanwending direk rondom die Eiland, geïdentifiseer as die voorgekeurde oplossing.

Die voorgestelde oplossing was ontwerp vir verskeie leeftye en uiterste vloedtoestande om die goedkoopste oplossing te vind. ‘n Ordegrote-kosteberaming vir die konstruksie van die struktuur is gemaak, wat die grondslag van vergelyking gevorm het met die koste wat voorsien is as geen vloedverdediging geïmplementeer word nie (die Niks-Doen alternatief). Die studie het bevind dat die voorgestelde oplossing ekonomies haalbaar raak as dit ontwerp word vir ‘n leeftyd van langer as 33 jaar. Die mees aantreklike oplossing was geïdentifiseer as die kombinasie van ‘n +3 m bo seevlak kruinhoogte bewapende dyk en ‘n +5 m bo seevlak kruinhoogte ruklipkeermuur.

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List of Abbreviations

GEC	Great Brak Environmental committee
CSIR	Council for Scientific and Industrial Research
HAT	Highest Astronomical Tide
LAT	Lowest Astronomical Tide
MHWS	Mean High Water Springs
MLWS	Mean Low Water Springs
MSL	Mean Sea Level
MLWN	Mean Low Water Neaps
MHWN	Mean High Water Neaps
SUH	Synthetic Unit Hydrograph
DRH	Direct Run-off Hydrograph
MCA	Multi-Criteria Analysis
EIA	Environmental Impact Assessment
EMP	Estuary Management Plan
WRP	Water Release Policy
FDM	Flood Defence Measure
IPCC	Intergovernmental Panel on Climate Change 2017
WMO	World Meteorology Organisation
SAWS	South African Weather Service
SED	Safety Evaluation Discharge

List of Symbols

Q	discharge
q	unit discharge
D_{n50}	equivalent cube length of median rock/median nominal diameter of rock
M_{n50}	mass of median armour unit
α	slope angle
H_{mo}	spectral significant wave height
H'_0	deep water significant wave height
H_S	significant wave height
H_{S0}	deep water significant wave height
K_S	shoaling coefficient
L_0	deep water wavelength
P	notional permeability of the structure
R_C	crest freeboard
$R_{u2\%}$	run-up level exceeded by 2 percent of the incident waves
s_{om}	fictitious wave steepness
s_{op}	deep water wave steepness
T_{m0}	spectral significant wave period
$T_{m-1,0}$	mean wave period
T_P	peak wave period
V_{max}	maximum overtopping volume
γ_b	reduction factor for influence of a berm
γ_h	reduction factor for influence of shallow-water conditions where the wave height distribution deviates from the Rayleigh distribution
γ_f	reduction factor for influence of surface roughness
γ_β	factor for influence of angle of incidence (β) of the waves
ξ	surf similarity parameter
ρ_w	mass density of water
ρ_r	mass density of rock
N_S	stability parameter
P_{ov}	probability of overtopping
Q_P	flood peak

1. Introduction

1.1 Background

The focus area of this study is the estuarine environment in the town of Great Brak. The town is situated between the towns of George (East) and Mosselbaai (West) on the Southern Coast of the Western Cape Province in South Africa. Great Brak is approximately 405 km from Cape Town and is accessible via the N2 National Road. The estuary (mouth) can be found at the following coordinates: 34° 3'23.84"S, 22°14'20.74"E. See Figure 1-1 for a visual depiction of the focus area.

Figure 1-1: Above: Locality of Great Brak in reference to Cape Town. Below: An initial look at the lower reaches of the Great Brak Estuary (Google Earth 2017)

As seen in Figure 1-1, the area surrounding the estuary is well-developed, consisting of residential and business properties. Some of these properties are on low-lying areas which experience inundation during the regularly occurring flood events. “The Island” is a residential development on an island, approximately 400 m by 230 m in size, in the lower reaches of the estuary, 250 m from the river mouth. Most of these developments lie under the +5 m MSL contour line and therefore experience regular flooding.

The river inflow into the estuary was reduced from a MAR of 37 million m^3 to a minimum of 1 million m^3 after the construction of the Wolwedans Dam upstream of the estuary. The dam was built to provide water for PetroSA at the Mosgas plant and for future domestic use by the towns of Mossel bay and Great Brak. This significant flow reduction inhibits the estuary mouth’s ability to remain open and the sand berm build up, due to prolonged mouth closure can cause hazardous water levels in the developed lower reaches of the estuary.

After 1 in 20-year flood in 2011 (See Figure 1-2), where the residences on the island experienced significant flood damage, the Council for Scientific and Industrial Research recommended that the construction of walls to protect low-lying property should be considered (Council for Scientific and Industrial Research 2011), seeing as larger future floods should be expected. This study will aim to investigate the danger of flooding and to propose a viable solution to increase safety for residents of low-lying properties.

Figure 1-2: The Island, in the Great Brak estuary, during the flood of 2011 (Source: Huizinga 2017)

1.2 Problem statement and study objective

The problem to be addressed in this study can be attributed to the existing low-lying property in and surrounding the estuary. These properties seriously limit the allowable water level of the estuary and drives the artificial mouth manipulations of the estuary. Flooding of these low-lying properties can be dangerous and can cost a lot.

The aim of the study is to identify and quantify the flooding components that contribute to extreme water levels in the Great Brak estuary and more specifically at the Island, in the lower estuary basin. A further objective is to investigate a conceptual design of an adequate flood defence measure for the Island.

The objectives of this study can be summarised as follows:

- Identify the relevant flood drivers (marine and fluvial) and methods to quantify these hazards.
- Investigate the estuary hydrodynamics, i.e. historical extreme water levels, mouth condition and berm geometry and the relationship between the upstream Wolwedans dam and the downstream estuary.
- Investigate possible measures for the alleviation of flood conditions at the residential development of the Island.
- Develop criteria for the performance evaluation of the possible flood defence measures.
- Investigate a conceptual design of an identified feasible flood defence, accompanied with an order of magnitude cost estimate.

The cost estimate will be based on the conceptual design of the identified preferred flood defence measure. The scope of work for this study does not include the detailed design of the flood defence measure, therefore the design water levels will not be calculated as per the best practice method which includes a detailed hydrodynamic modelling process of the estuary and the interaction of extreme river run-off, extreme ocean waves and still water level.

1.3 Method statement and thesis layout

This section provides more detail on the work carried out in this study to meet the objectives set out in Section 1.2, and includes a brief overview on each section.

In the Literature study (Section 2) which was conducted during this research project, the definition and hydrodynamics of South African estuaries will be examined to help understand fundamental characteristics of the South African estuary and the nature of its hydraulic response to fluvial and marine driven flooding. The section goes on to describe methods to be used in the quantification of the fluvial and marine driven flooding components as well as methods prescribed to investigate possible water levels in conventional estuaries. Further, the section considers climate change and its effects on estuaries and on floods (fluvial and marine).

In the Situation assessment of the Great Brak estuary (Section 3) the catchment characteristics that will be used to quantify the fluvial and marine flooding components will be defined. The section will briefly describe the history of the estuary and the events which led to the formation of the Estuary Management Plan as well as a description thereof, which governs the artificial mouth manipulations and monitoring schemes in the estuary. Past studies conducted on the Great Brak estuary, e.g. hydrological studies and river modelling, are investigated alongside historical flood events and water levels to establish a baseline of possible extreme water levels in the lower estuary basin.

After the flood drivers are identified, potential flood defence measures are described in Section 4. This section focuses on possible measures of alleviating flood conditions at the Island due to high water levels and possible direct wave attack. The section concludes by establishing performance evaluation criteria by which the possible flood defence measures will be evaluated to find a preferred option.

Section 5 will quantify the fluvial and marine flooding components using the methods described in Section 2. A flood hazard assessment and vulnerability mapping will be utilised to identify the vulnerable portions of the Island perimeter. The potential flood defence measures will be evaluated in Section 6 and by means of an objective Multi-Criteria Analysis, the preferred option will be identified. The preferred option will then be designed on a conceptual basis which will be accompanied by an order of magnitude cost estimate. The cost of construction of the proposed flood defence measure will be estimated as basis for comparison against the costs foreseen for the Do-Nothing alternative, which will give a rudimentary indication whether the proposed flood defence measure can be economically justified.

Conclusions from this study and recommendations for further design of the preferred flood defence measure will be described in Section 7.

2. Literature Study

2.1 Introduction

This chapter will discuss all relevant literature for understanding the hydrodynamics and processes in the estuarine environment.

There are approximately 300 estuaries identified around the South African coastline. The variability of annual precipitation in the country has a direct influence on the estuarine characteristics. There are three main biogeographic regions, around the South African coast, wherein the estuaries can be sorted. These regions are the Cool-Temperate, Warm-temperate and the Subtropical regions. The subtropical estuaries are located mainly on the North – Eastern coast of South Africa and typically experience the highest rainfall. The Warm-temperate region is located on the Southern coast of South Africa and experiences medium to high rainfall whereas the Cool-temperate region of the Western coast mainly experience low rainfall in a year (van Niekerk 2015). See Figure 2-1 for a graphical representation of the three biogeographical regions.

The literature review will attempt to define what an estuary is in the South African context and to cover the theories surrounding estuarine classification. The literature regarding estuary hydrodynamics will

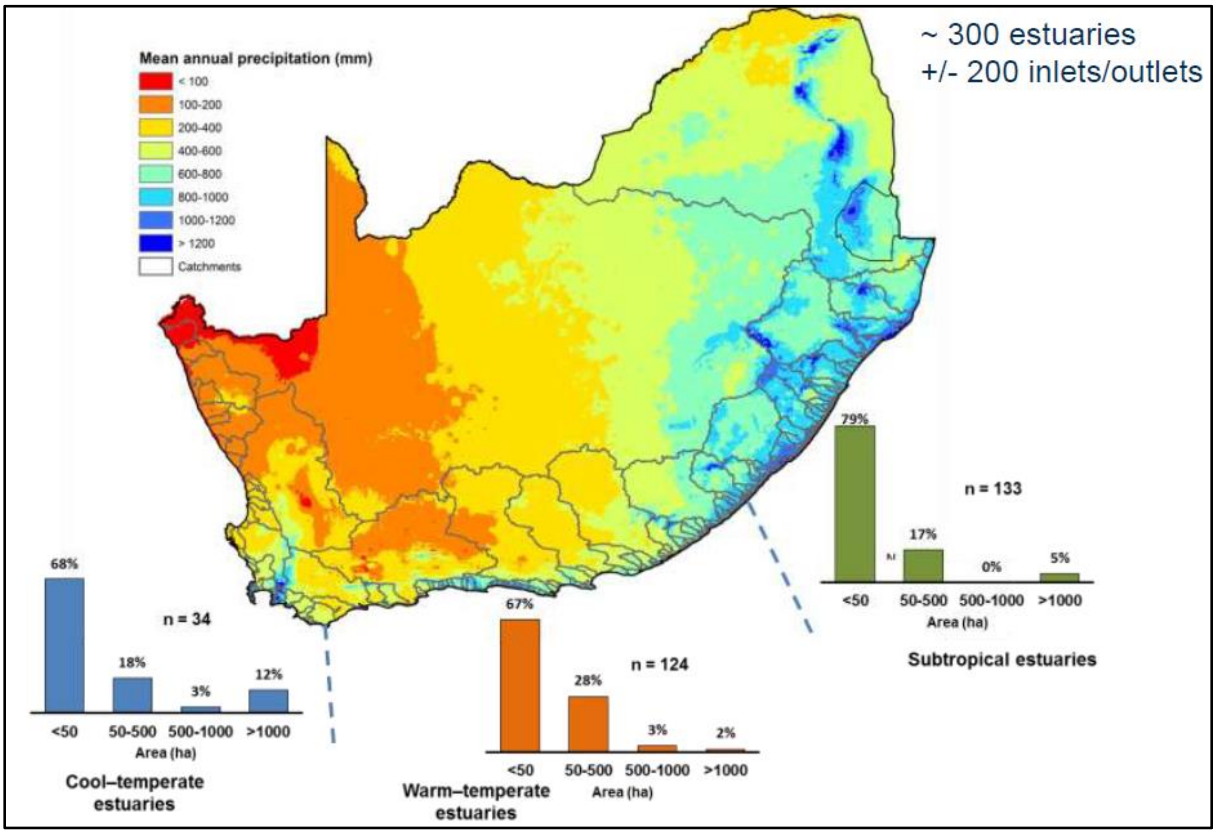


Figure 2-1: Climatic regions of South Africa and information regarding the estuaries located in these regions (Source: van Niekerk 2015)

also be covered alongside some known external factors that pose a threat to estuarine health. The hydrodynamics, extreme waves and rainfall expected at Great Brak Estuary will be described. Climate change will also be taken into consideration. The rising of sea levels and the influence of climate change on regional rainfall patterns are some of the factors to be explored.

2.2 Definition of an Estuary

The definition of an estuary is important to understand as context for this study. In international literature, an estuary is defined as ‘a semi-enclosed coastal body of water which has a free connection with the open sea and within which sea water is measurably diluted with fresh water derived by land’ (Pritchard 1967).

South African Estuaries differ greatly from the estuaries in the Northern Hemisphere, which is mainly observable in the size of the estuary. South African estuaries are smaller than estuaries in the Northern Hemisphere seeing as the run off varies greatly, from flood events to the extreme of no river inflow. The variability in the run off in conjunction with the typical high energy coast line of South Africa led to a number of different definitions for South African estuaries.

An amendment to the National Environmental Management: Integrated Coastal Management Act (24 of 2008) in 2014 updated the legal definition of an estuary as a body of surface water-

- ❖ That is permanently or periodically open to the sea;
- ❖ In which a rise and fall of the water level as a result of the tides is measurable at spring tide when the body of surface water is open to the sea; or
- ❖ In respect of which the salinity is higher than fresh water as a result of the influence of the sea, and where there is a salinity gradient between the tidal reach and the mouth of the body of surface water

The most comprehensive definition for the South African estuary according to van Niekerk, Taljaard e*t al* (2012), is the definition outlined in the South African National Report (CSIR 1992) for the United Nations Conference on the Environment and Development held in Rio de Janeiro, June 1992. It reads as follows:

‘In South Africa, an estuary is considered to be that portion of a river system which has, or can from time to time have, contact with the sea. Hence, during floods an estuary can become a river mouth with no seawater entering the formerly estuarine area. Conversely, when there is a little or no fluvial input an estuary can be isolated from the sea by a sandbar and become a lagoon which may become fresh, or hypersaline, or even completely dry.’

South African estuaries can further be classified based on their hydrodynamic features. Whitfield (1992) classifies South African estuaries based on its dominant mouth state. All estuaries in South Africa can be classified in one of two categories, namely: permanently open or temporarily open/closed. This can be done irrespective of the individual estuarine system's climate, topography and catchment geography.

- **Temporarily open/closed estuaries (TOCE):** These types of estuaries are open or closed to the ocean environment, depending on river flow. This category includes estuarine lakes and some river mouths. A sand berm develops across the mouth of the estuary during periods of little to no river flow in conjunction with longshore sediment transport (Whitfield 1992). When the basin then fills up to a higher water level, the sand berm is breached and the mouth will be flushed open again, resulting in the removal of significant amount of sediment. These estuaries generally have small river catchments (< 500 km^2) as well as small tidal prisms (< $1{\times}10^6\ m^3$) (Whitfield 1992). About 75 % of the estuaries in South Africa have been classified as TOCE (Van Niekerk, L. and Turpie, J.K (eds), 2012). The Groot Brak estuary is an example of a TOCE (Whitfield, Bate 2007).
- **Permanently open estuaries (POE):** These types of estuaries are permanently open to the ocean environment. This category includes estuarine bays and some river mouths. These estuaries have moderate tidal prisms (1-10 $\times 10^6\ m^3$), generally large catchment areas (>500 km^2) and a high run-off (Whitfield 1992). The availability of sediment and shelter from high wave energy conditions plays a significant role in maintaining open mouth conditions

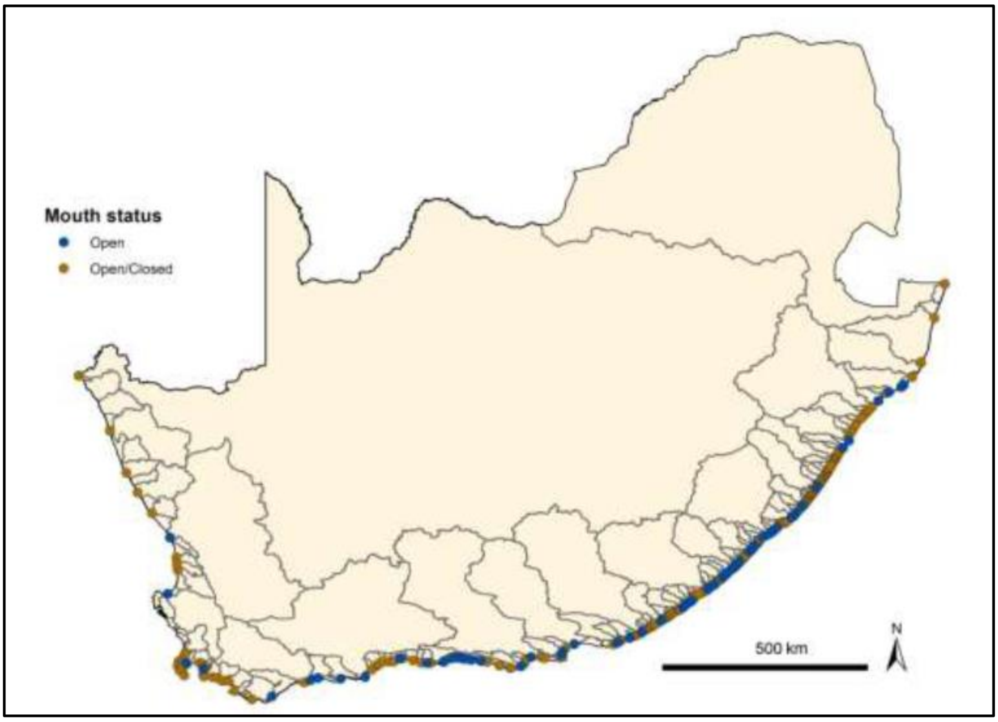


Figure 2-2: Location of permanently open (indicated in blue) and temporarily open/closed (indicated in brown) estuaries around South Africa (Source: (Van Niekerk, L. and Turpie, J.K (eds), 2012)

(Whitfield, Bate 2007). About 25 % of the estuaries in South Africa is classified as POE (Van Niekerk, L. and Turpie, J.K (eds), 2012). The Breede estuary is an example of a POE.

See Figure 2-2 for a spatial layout of the abovementioned types of estuaries located in South Africa.

2.3 Estuary Hydrodynamics

The hydrodynamics of South African estuaries will be discussed in this section. It is necessary to understand these important abiotic characteristics to develop adequate flood defence measures for the Great Brak estuary.

2.3.1 Tidal variation

Tidal variation is a direct result of the gravitational forces between the earth, sun and the moon. The elliptical orbits of the moon around the earth and the earth around the sun ensures that there are a maximum and minimum gravitational pull during each orbit, therefore resulting in the tides occur roughly twice daily, i.e semi – diurnal. When the sun and the moon align in space around the earth, either combining their gravitational forces or opposing each other, two different tides occur, namely Spring and Neap tide. Spring tides occur at new and full moon and bring a large tidal variation whereas neap tides occur at first and third quarter and result in a very small tidal variation (Beck, *et al* 2004).

The tidal range varies around the world, and has roughly been classified as follows:

- Microtidal: 0 – 2 m
- Mesotidal: 2 – 4 m
- Macrotidal: > 4 m

The marine climate of the South African coast has been described to be wave-dominated, due to its generally low tidal ranges and high wave energy (Cooper 2001). With a tidal range of less than 2 m, the nearly 300 functional estuary systems around the South African coast has been defined as predominantly microtidal. The systems are shallow (average depth 2 – 3 m) and highly dynamic (Van Niekerk, L. and Turpie, J.K (eds), 2012).

From measured data along the coast, the tidal ranges of different types of estuaries can be generalized. For permanently open estuaries, the tidal range is typically greater than 1.5 m (e.g. Berg, Olifants), and for large temporarily open/closed estuaries the tidal range is between 0.5 and 1.5 m (e.g. Groot Brak, Seekoei) (Van Niekerk, L. and Turpie, J.K (eds), 2012).

2.3.2 Tidal prism

The tidal prism or the inter-tidal volume of an estuary is a widely-used concept and several different definitions thereof exist. Most definitions refer to the volume of water entering and leaving the estuary on flow and ebb tides. Some definitions from the literature are cited as follows:

i. The volume of water in an estuary of tidal inlet between the water levels at high tide and low tide (Luketina 1998).
ii. The volume of water leaving an estuary on the ebb tide (Davis Jr, FitzGerald 2009).
iii. The volume of water that is drawn into the bay, from the ocean through the inlet, between low water slack and the next high water slack, i.e. during flood (Bruun 2013).

The tidal prism can be quantified as a function of the open water area of the estuary and the tidal range (Davis Jr, FitzGerald 2009, Lakhan 2003). And can be denoted as:

$$P = H_{tide} \cdot A \qquad \textbf{2-1}$$

Where:

P : The tidal prism

H_{tide} : The tidal range

A : The average surface area of the basin

2.3.3 Mouth state

The estuary mouth experiences a variety of major forces from the river and the ocean. These forces help to either keep the mouth open or can help close it. The opening and closing mechanisms will be discussed in this section. The mouth state of an estuary can experience multiple openings and closures in a year and is highly sensitive to the seasonal weather patterns. During mouth closures the ability to access the ocean via boat will be inhibited and the general water quality in the estuary may deteriorate.

According to research done by the CSIR, a third mouth state was identified: the semi-closed state (Whitfield, Bate 2007). This state occurs when the estuary mouth is nearly closed with only a narrow and shallow surface drainage channel allowing water to trickle out to the ocean. This mouth state is described to be usually perched and too shallow to allow tidal intrusion. This phenomenon is said to occur at many smaller estuaries along the South African coast.

The closing of an estuary along the South African coastline can be attributed to the following major forces (van Niekerk, *et al* 2012):

❖ Wave energy, in conjunction with the following variables:

    - Beach slope
    - Berm height
    - Width of breaker zone
    - Median grain size
- Sediment availability, both marine and fluvial

Open mouth conditions are caused by:

- Tidal flows
- River inflow

2.3.3.1 Opening mechanisms

The river inflow of the estuarine system is the most important mechanism in maintaining open mouth conditions of South African estuaries. In the smaller estuaries it is the only mechanism opening the mouth, however, in larger estuaries the tidal flow assists in maintaining open mouth conditions (van Niekerk, *et al* 2012). Permanently open mouth conditions are predominantly observed in large (> 100 ha) systems, seeing as the tidal prism is large enough, where smaller estuaries with smaller tidal prisms will close during neap tides.

Some geological and marine structures can facilitate the prolonged opening of an estuary mouth in smaller to medium-sized estuaries. Rocky headlands surrounding the tidal inlet, a reef system in the surf-zone in front of the inlet and a rocky shelf running below the surface of the ocean are all examples of natural features to mitigate the closure of the mouth (van Niekerk, *et al* 2012).

2.3.3.2 Closing mechanisms

As stated above, the two major forces that will influence the closure of the estuary mouth are the wave energy at the estuary mouth and the available sediment budget of the fluvial and marine system.

According to van Niekerk, Taljaard et al (2012), the **breaker zone width**, **beach slope**, **median grain size** and the **berm height** of the beach are all parameters that can accurately describe the wave energy situation at an estuary mouth.

High-energy wave conditions can be expected when a steep **beach slope** is observed at the estuary mouth. In general, the wave energy is directly proportional to the steepness of the beach slope. Higher wave energy will result in a higher suspended sediment load that is available to enter the estuary mouth thus increasing the possibility of mouth closure.

Coarse **grain size** sediment of the beach close to an estuary mouth indicates the presence of high energy wave conditions. This indicates the presence of sufficient energy for sediment transport, thus causing sedimentation in the estuary mouth.

Another indication of high energy waves can be the **width of the breaker** zone. Generally, the breaker zone width is inversely proportional to the available wave energy at the shore. A narrow breaker zone indicates higher wave energy at the shoreline, seeing as the energy is not adequately dissipated as in the case of a wide surf-zone.

The **berm height** of the beach is directly proportional to the wave energy at the location. Higher berm heights (>3 m MSL) indicates high energy coastline, seeing as the waves can run-up high enough to still deposit sediment on the berm.

The sediment budget is the other major factor in the closure of estuary mouths in South Africa. The **longshore sediment transport**, **availability of sediment in proximity to the estuary mouth or resident sediment** and **sediment carried down from the catchment** are the influencing factors on the sediment budget.

The **longshore sediment transport** rate indicates the presence of available sediment needed for mouth closure. The proximity of the longshore current to the estuary mouth also influences the possibility of mouth closure. Physical obstructions to the longshore current can also affect the availability of sediment for mouth closure, for example a rocky headland may deflect the longshore current away from the estuary mouth.

The size and width of a beach adjacent to the mouth gives an indication of the volume of **resident sediment** available for mouth closure. Long stretches of wide beaches have significantly more sediment available for mouth closure in comparison to a predominantly rocky coastline with pocket beaches.

The **catchment sediment** can also supply sufficient sediment for mouth closure. A flood event generally scours and flushes the sediment out of an estuary. These sediments will accrete outside the mouth of the estuary where it can be picked up and carried into the mouth of the estuary causing closure.

The closing of the estuary mouth is a combination of all the above and cannot be attributed to a single factor. Ranashinge et al (1999) described two tidal inlets' closing mechanisms applicable to small estuaries on micro-tidal and wave dominated coasts, like that of South Africa. These two mechanisms are described below and is depicted in Figure 2-3.

i. <u>Mechanism 1: The interaction of the inlet current and the longshore current.</u>

The longshore sediment transport current is disrupted by the tidal inlet current. A shoal will form upstream of the estuary mouth, due to the sediment deposition brought on by the diversion of the ebb current by the longshore current. When high river flow periods are experienced the shoal may be flushed away and the inlet will stay open. In low river flow periods, the shoal may continue to grow and thus eventually close the mouth of the estuary (Ranasinghe, *et al* 1999).

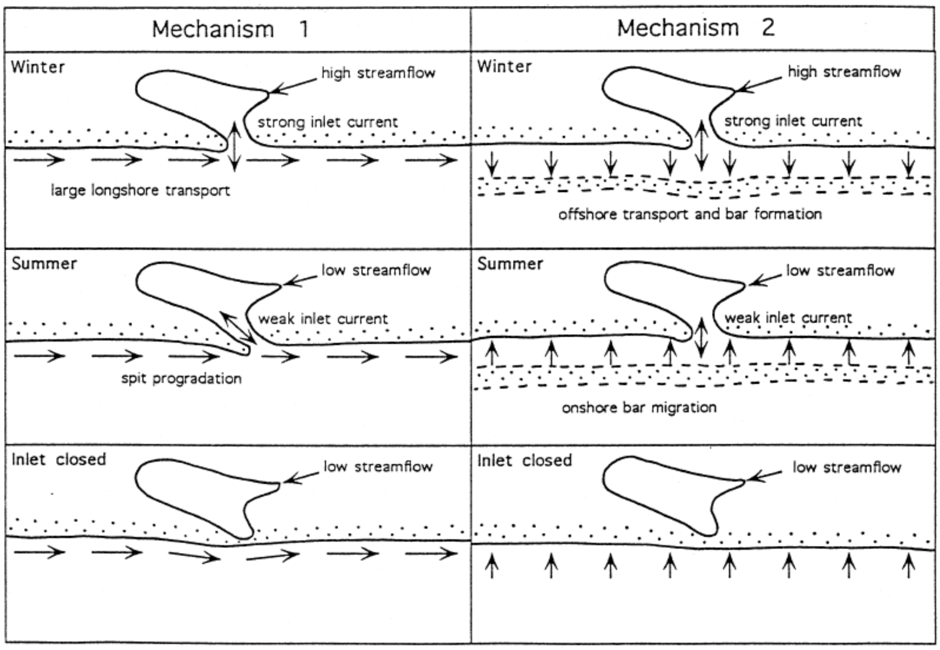


Figure 2-3: Closure mechanisms (Source: Ranasinghe, Pattiaratchi et al. 1999)

ii. Mechanism 2: The interaction between the inlet current and onshore sediment transport

This mechanism will dominate in conditions where the tidal inlet current is small, resulting in smaller tidal prisms and micro – to mesotidal environments. Stormy conditions (winter) will cause sand eroded from the surf-zone and beach to accumulate offshore, forming a sand bar at the breaker position. When the storms subside, long-period swell waves will dominate and start transporting the sediment onshore, where it will interact with the tidal inlet. When the ebb current of the inlet is small, during low flow periods, the continuous cross-shore transport will cause the closure of the mouth.

2.3.4 Ebb and flood channels

Estuaries generally exhibit two distinct flow channels. The ebb- and flood flow channels develop due to inertia effects (Whitfield, Bate 2007). The flood tidal channel is normally the most direct channel into the estuary and is utilised during the flood tide with a high flow rate. A large part of the sediment transported into the estuary is deposited just inside the inlet, where a flood tidal delta is formed.

The ebb-tidal flow channel is normally smaller and meanders around the flood tidal delta. This occurs due the much lower velocity of outflow considering that the period of outflow is longer. The well-protected ebb channel plays an important role in maintaining open mouth conditions.

The ebb and flood flow channels often cross and this is where shoaling can take place. During the ebbing tide, sand is deposited in the flood channel and vice versa. These channels can cross on several locations and are not always stable, thus they tend to shift with time. This leads to shoaling in several places and can lead to the mistaken conclusion that the estuary is experiencing sedimentation (Beck, *et al* 2004).

2.3.5 River inflow and catchment size

The characteristics of the river catchment plays a large role in the hydrodynamics of the estuary. The size of the catchment and the run-off is not necessarily linked, due to the arid South African climate. However, POEs tend to have larger catchments with noteworthy river flow throughout the year while TOCEs generally have smaller catchments with strong seasonal variation in run-off (Whitfield 1992).

The river inflow or baseflow has a major influence in maintaining open mouth conditions and is frequently the only driving force behind an open mouth state. This is generally the case in smaller estuaries, whereas in larger areas, the tidal influence can help maintain open mouth conditions (Whitfield, Bate 2007). The baseflow to maintain open mouth conditions vary greatly along the South African coast and is reliant on the wave conditions near the mouth and the availability of sediment.

TOCEs tend to be sensitive to flow modification. Changes in frequency and duration of mouth closure usually follows flow manipulation. In severe cases, the reduction of freshwater inflow to an estuary can cause the permanent closure of the mouth. As in the case of the Isipingo Estuary, where a reduction in MAR from 102 $\times 10^6$ m^3 to 3 $\times 10^6$ m^3 led to the almost permanent closure of the estuary mouth (Whitfield, Bate 2007).

2.3.6 Estuary size and bathymetry

The estuary size normally influences the time of the open mouth conditions of temporarily open estuarine systems. In larger systems, > 150 ha, the tidal interaction of the estuary can maintain open mouth conditions for longer even when low river inflow is experienced. Despite the large size of estuarine lakes, they are an exception to this rule, as they may close due to an array of possible reasons, e.g. high evaporation rates, high sediment availability, extended low flows, high wave energy (Whitfield, Bate 2007).

In estuaries, smaller than 150 ha, often termed medium sized estuaries, tidal flow is said to have a significant influence on maintaining an open mouth condition during spring tide, whereas it will close during neap tides. This can be observed in the Great Brak and Seekoei systems (Whitfield, Bate 2007). The tidal prism is thus too small during low flow periods to maintain an open mouth.

The semi-closed mouth state can generally be observed in small estuaries. In smaller estuaries, lower river inflow is required to maintained the desired equilibrium state between the closing forces and the river inflow that causes the rising water level, which is needed to maintain this mouth condition. According to Whitfield and Bate (2007), this mouth condition only occurs in estuaries < 100 ha and < 2 km long.

The bathymetry of an estuary has a significant influence on the water circulation. Deeper estuaries are more likely to develop stratification directly behind the sand berm. Stratification is when an estuary become poorly vertically mixed, with the freshwater layer separates from the underlying saltwater layer. Stratification is not likely to develop in smaller shallower estuaries, due to the limited storage area.

2.3.7 Estuary basin model

An estuary is subjected to marine and river discharge-induced water motion. The dominant hydraulic loadings to consider depends on the locality of the site considered. The Island in the Great Brak estuary is in the lower reaches of the estuary basin and may experience marine and river dominated flow regimes. The flow of water in estuaries accord with physical laws and upstream and downstream boundary conditions govern the flow regime. The estuary geometry, the seaward tidal level and the river discharge are all boundary conditions an estuary must abide to.

Flow condition computation is based on fundamental principles like the conservation of mass and momentum. With a combination of these principles, boundary conditions and experimentally determined parameters most flow problems can be solved (CIRIA 2007). Tidal currents, density-currents, longshore currents, wind-induced currents and river discharge are the principal currents to consider in the design phase of any structural works in an estuarine environment. The tidal intrusion and river discharge are commonly the most significant, if extreme conditions are considered.

The water level, flow velocity and discharge can be estimated using a basin storage model if the relative length of the estuary is short compared to the length of the tidal wave ($L_b/L < 0.05$). The definition of the basin model parameters can be viewed in Figure 2-4. The Rock Manual (2007) distinguishes between two cases relating to the inlet geometry. Case 1 – where there is no appreciable constriction of the estuary mouth and Case 2 – where there is vertical closure of the estuary mouth.

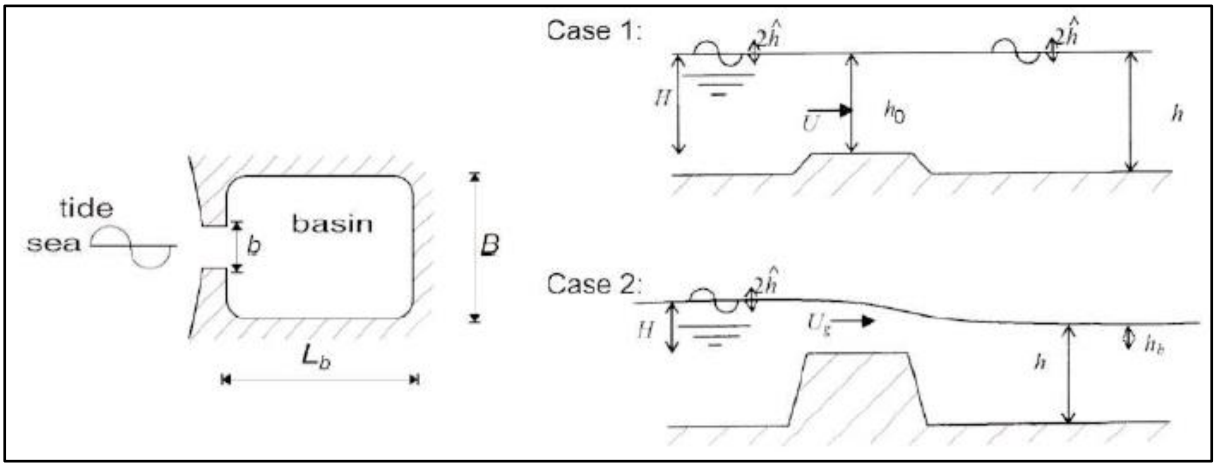


Figure 2-4: Estuary basin model definition sketch (Source: CIRIA 2007)

<u>Case 1</u>

Case 1 is defined as the case where the ratio b/h_b is sufficiently large. The discharge through the tidal inlet due to the vertical tide inside the basin can be determined by Equation 2-2.

$$Q(t) = BL_B \frac{dh}{dt} \quad \text{2-2}$$

Where $Q(t)$ = tidal discharge (m³/s) and h = the water level in the estuary. In the case of a sinusoidal tide, Equation 2-2 becomes Equation 2-3.

$$Q(t) = \frac{2\pi}{T} BL_b h \sin\left(\frac{2\pi t}{T}\right) \quad \text{2-3}$$

Where h = tidal amplitude (m), t = time after the beginning of the tide (s) and T = tidal period (s). The cross-sectional mean velocity, U, through the estuary mouth can then be determined by Equation 2-4.

$$U = \frac{Q}{bh_0} \quad \text{2-4}$$

Where h_0 = the water depth in the gap.

Case 2

The second case is defined as where a notable vertical closure can be seen at the estuary mouth. The tidal discharge and flow velocity will decrease and depend on the water levels in the basin and on the seaward side. The water level response in the estuary will then be determined by a simple model described by Equation 2-5.

$$BL_b \frac{dh}{dt} = h_0 b \sqrt{2g(H_{ocean} - h_b)} + Q_{river} \quad \text{2-5}$$

Where Q_{river} is the river discharge (m³/s) into the basin, H_{ocean} = the seaward water level above crest level and h_b = water level in estuary basin above the crest of the closure dam. When the flow of water is into the basin due to tidal influence, the mean flow velocity can as a first estimate be determined by Equation 2-6.

$$U_0 = \sqrt{2g(H_{ocean} - h_b)} \quad \text{2-6}$$

2.3.8 Inlet geometry approach

In the estuary basin model discussed in Section 2.3.7, a fixed inlet geometry is assumed in order to estimate water levels in the estuary, discharge regimes and outflow velocities. This is not always possible, as most estuaries around South Africa and specifically the Great Brak estuary is constricted by a sand-berm that has a complex relationship with the river run-off and the tidal influence. The increase of discharge through the mouth inlet, fluvial or marine driven, will lead to scouring of the fine sediment present and ultimately a larger outlet geometry will be the effect. The previous causes larger discharges and faster drainage of the basin area. Thus, water levels may be overestimated if an overly conservative outlet geometry is assumed.

The Rock Manual (2007) describes two methods of calculating a unit discharge (m³/s/m) of water flowing over rockfill closure dams. The horizontal closure method is used when a closure dam is built across a river from the sides where the vertical closure method is used when a dam is built up from the river bed. The vertical closure method of estimating the unit discharge over the vertical constriction was chosen to approximate the flow out of the estuary mouth. The unit discharge can then be multiplied by a width of flow to obtain the discharge, Q (m³/s).

The nominal diameter of the rocks, D_{n50} (sediment in this case), the relative density, Δ, and the tail water depth, h_b, is used to determine the governing flow regime by means of a tail water parameter. The nominal diameter of the sediments present in the Great Brak estuary was found to be 352 μm (Hugo. 2013) and will thus cause the tail water parameter to be larger than 4, thus low dam flow is the governing flow regime. The mouth berm is essentially treated as a broad crested weir, where the unit discharge can be calculated by Equation 2-7 for sub-critical flow and by Equation 2-8 for super-critical flow.

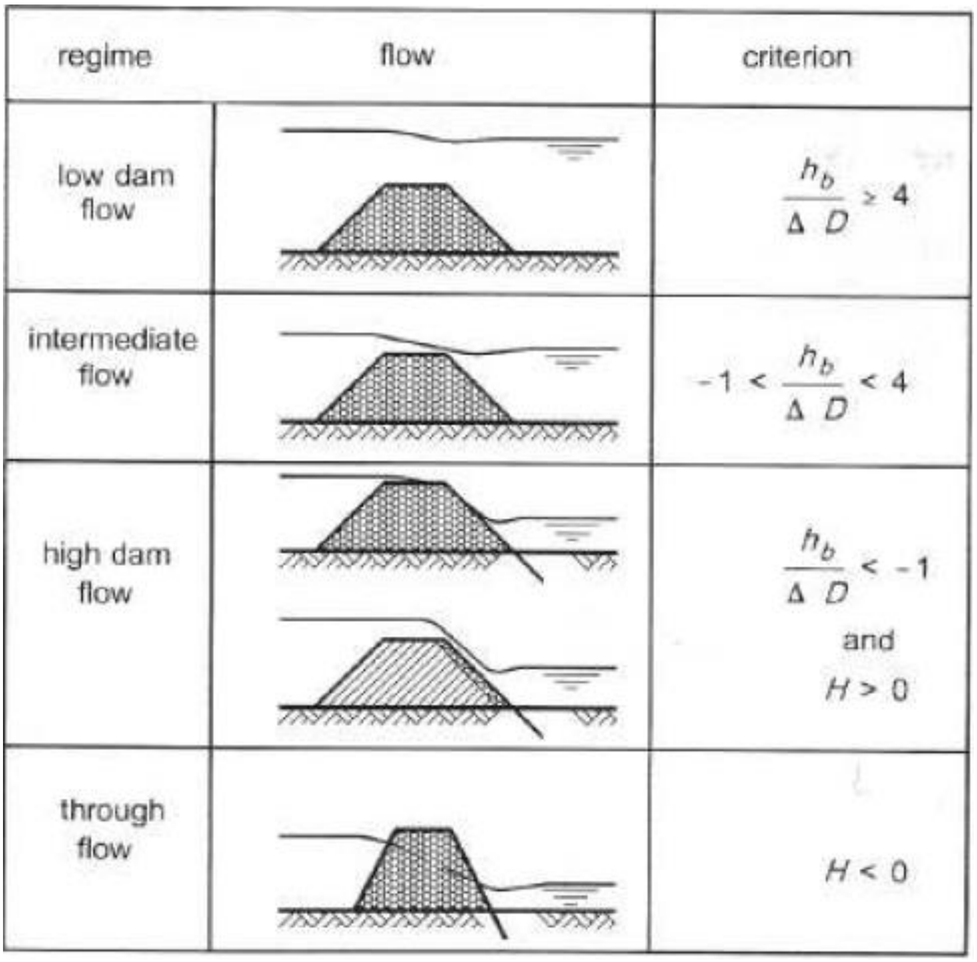


Figure 2-5: Typical flow regimes for crest flow (Source: CIRIA 2007)

$$q = \mu h_b \sqrt{2g(H - h_b)} \quad \text{2-7}$$

$$q = \mu \frac{2}{3} \sqrt{\frac{2}{3}(gH^3)} \quad \text{2-8}$$

Where:

q = Unit discharge (m³/s/m)

μ = Discharge coefficient (-), chosen as 1.1 for low dam, non-porous and rather smooth

h_b = Downstream water level relative to crest level (m)

H = Upstream water level above crest level (m)

Then the mean depth flow velocity, U_0 (m/s), can be calculated by using Equation 2-9.

$$U_0 = \frac{q}{h_0} \quad \text{2-9}$$

Where, h_0 is the minimum flow depth on the crest of the closure dam. To calculate whether the flow is super-, or sub-critical, the criterion of Equation 2-10 is used.

$$\begin{aligned} Sub-critical&: \ h_b > \frac{2}{3} H \\ Super-critical&: \ h_b < \frac{2}{3} H \end{aligned} \quad \text{2-10}$$

This method still cannot account for a variable depth of the inlet geometry. Which will be mitigated by assuming various initial values for width and depth. The inability to adequately describe the scouring of the estuary inlet channel will ultimately lead to overestimation of possible water levels in the estuary. The applicability of this model to South African TOCE estuaries is limited and will prove to be an inadequate method to determine possible return period water levels in the estuary. The complexity of the estuary morphological and hydrodynamic responses can possibly be described by numerical modelling software packages, which should be attempted in the design phase of a potential flood defence measure.

2.3.9 Relationship between initial water level and discharge

From Section 3.6, the height at which breaching of the Great Brak estuary mouth was undertaken increased from +1.6 m MSL to +2.0 m MSL. The height increase was found to have a direct relationship with the flushing efficiency and prolonged open mouth conditions. It is therefore beneficial to breach at the highest point possible. The height at which the estuary is breached at present, is controlled by low-lying development in the estuary basin. From Figure 2-6 can be seen how the maximum discharge increases with the increase in initial water level before breaching.

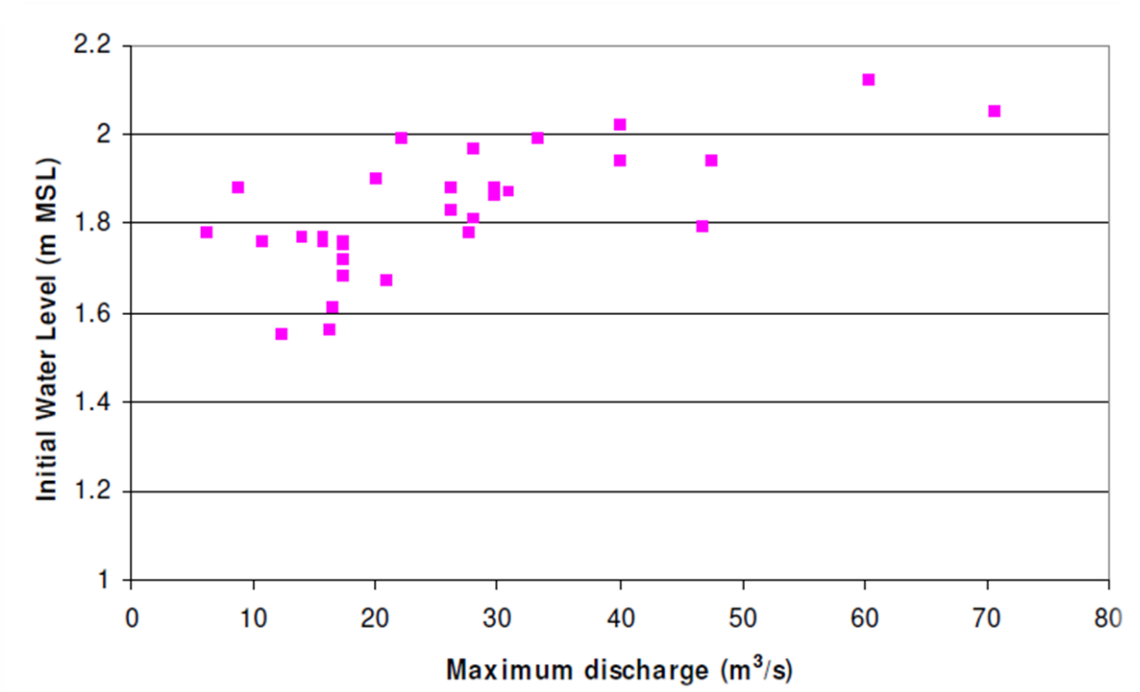


Figure 2-6: Observed peak discharges versus initial water level in the estuary basin during breaching events (Source: Beck et al 2005)

The application of a flood defence measure around the Island, will make it possible for higher water levels in the estuary to be achieved, without causing extensive inundation to the low-lying properties on the Island. This is advantageous to the flushing efficiency of the estuary during planned breaches, which will cause the inlet to be breached open wider and deeper and subsequently delay closure and in turn save water from being used in flush releases from the Wolwedans Dam. The flood defence measure may also replace the water-release assisted emergency breaching protocol, described in Section 3, which will save more water. The cost of water estimated to be saved by implementing a flood defence measure around the Island will be considered later in the study when assessing the economic feasibility of the proposed measure.

2.4 Pressures on estuaries

Estuaries, being the sensitive systems that they are, experience a lot of pressures from human intrusion as well as some natural processes. This section aims to highlight these pressures. Estuarine health and biodiversity can be threatened by the following factors (Van Niekerk, L. and Turpie, J.K (eds), 2012):

- Pollution
- Exploitation of living resources
- Flow modification
- Urban development
- Mouth manipulation
- Climate change (discussed in Section 2.5)

Pressures relevant to this study will be expanded on in this section. The Wolwedans Dam, upstream of the Great Brak estuary, has reduced the natural run-off significantly and therefore experiences flow modification. The estuary mouth berm of Great Brak has been artificially breached for various reasons through the years (See Section 3.2.3) and therefore pressures from mouth manipulation is deemed to be a relevant issue. Climate change effects will be discussed in subsequent sections.

2.4.1 Flow modification

As stated in Section 2.3.3, the freshwater inflow into an estuary has a direct influence on the mouth dynamics of the estuary. Flow modification refers to the reduction and increase in freshwater inflow into an estuary. A reduction in river inflow can be caused by direct abstraction, dam development and the cumulative effect of smaller farm dams (Van Niekerk, L. and Turpie, J.K (eds), 2012). The increase in river inflow can be caused by Waste Water Treatment Works, inter-basin transfer schemes and catchment hardening.

The closure of mouths, normally permanently connected to the ocean, is a direct result of freshwater flow reduction. This was the case at the Kobonqaba Estuary and at the Uilkraals Estuary, situated in the Eastern Cape and Western Cape respectively, where the mouth closed for the first time in 2010 (Van Niekerk, L. and Turpie, J.K (eds), 2012). The mangrove forest in the Kobonqaba Estuary experienced a significant die-back during the mouth closure. Mouth closure can also cause an increased water level which can cause drowning of trees not normally submerged. The increase of freshwater inflow may help in keeping the mouth of an estuary open that normally close under low flow conditions. This can prevent the regular back-flooding and aid in the increase in habitat of an estuary. E.g. the Waste Water Treatment Works near the Eerste Estuary.

The impacts of flow modification on the Great Brak estuary was mitigated by assessing the Ecological Water Requirement of the estuary and adopting a Water Release Policy into the Estuary Management Plan.

2.4.2 Mouth manipulation

The manipulation of the estuary mouth can lead to the change in type of estuary, e.g. from a TOCE to a POE. These manipulations can occur in different forms: redirecting of the outlet, artificial breaching, and channelization. The need for mouth manipulation often arises from inadequate development (e.g. below the flood line) inside the estuarine functional zone.

Backflooding of the upstream properties usually drives the necessity of artificial breaching. This type of manipulation is the most relevant and is often driven by an increase in mouth closure, which might be a by-product of flow manipulation. Flow manipulation as an impact on estuaries was discussed in Section 2.4.1. The artificial breaching of an estuary mouth can lead to a change in mouth dynamics which can alter the sediment dynamics, water quality and salinity of the estuary, further causing changes in ecological productivity.

According to van Niekerk and Turpie (2012), the backflooding water level during closed mouth conditions, which is higher than the high-water level, was not recognised by any South African legislation, up until 2014; only the high-water mark was recognised. This often led to authorities being forced to make the decision to breach the estuary berm. This was however amended in Act No.36 of 2014: National Environmental Management: Integrated Coastal Management Amendment Act (2014), where the definition of the "high-water mark" was changed to exclude any water level reached within an estuary closed to the sea.

The artificial breaching protocol of the Great Brak mouth berm forms part of the Estuary Management Plan and is driven by a weighted score decision support system based on the monitoring of the biotic and abiotic variables found in an estuary as well as planned breaching. See Section 3.2.3 for a description of the Estuary Management Plan.

2.5 Climate change

The estuarine environment is subject to overland flooding and is vulnerable to extreme coastal events, being the interface of freshwater and the ocean. It is therefore necessary to understand the impact of climate change on the estuarine environment and what it would mean for the existing development within the functional zone of the estuary.

The climate change cause-and-effect is subject to ongoing research and updating global climate forecast models. The Intergovernmental Panel on Climate Change (IPCC) is an organisation established in 1988 by the United Nations Environmental Programme (UNEP) and the World Meteorological Organisation (WMO) (IPCC - Intergovernmental Panel on Climate Change 2017). The IPCC does not conduct any research nor do they monitor any climate related data; they merely exist to provide the world with a scientific view on the current state of knowledge surrounding climate trends.

In the latest instalment of the IPCC, the Fifth Assessment report (AR5), global warming is assessed as *virtually certain* (99% - 100% likely). The last three decades was *very likely* (90% - 100% likely) the warmest 30-year period of the last 800 years in the Northern Hemisphere. Globally, temperatures of combined land and ocean surface increased linearly by 0.85 ℃ over the period of 1880 – 2012 (Pachauri *et al.* 2015). The warming of the ocean accounts for more than 90% of the energy accumulated between 1970 and 2010 and dominates the increase in energy stored in the climate system. The result of ocean warming is the thermal expansion of the water mass, thus raising the mean sea level. Thermal expansion of the ocean with glacial mass loss explains approximately 75% of the observed increase in global mean sea level (Pachauri *et al.* 2015).

According to research on climate change impacts on South African port and maritime infrastructure by Rossouw and Theron (2012), there is a progressive trend in the mean annual significant wave height (H_{m0}) that accompany individual winter storms. An increase of 0.5 m over a period of 14 years (1994 - 2008) was observed (See Figure 2-7). This was however deemed not to be indicative of the long-term trend regarding sea level rise, but rather an increase in storminess during the winter periods (Rossouw, Theron 2012). According to the IPCC, extreme sea levels, as experienced in storm surges, have *likely* increased since 1970, attributing the increase to a rise in mean sea level rise.

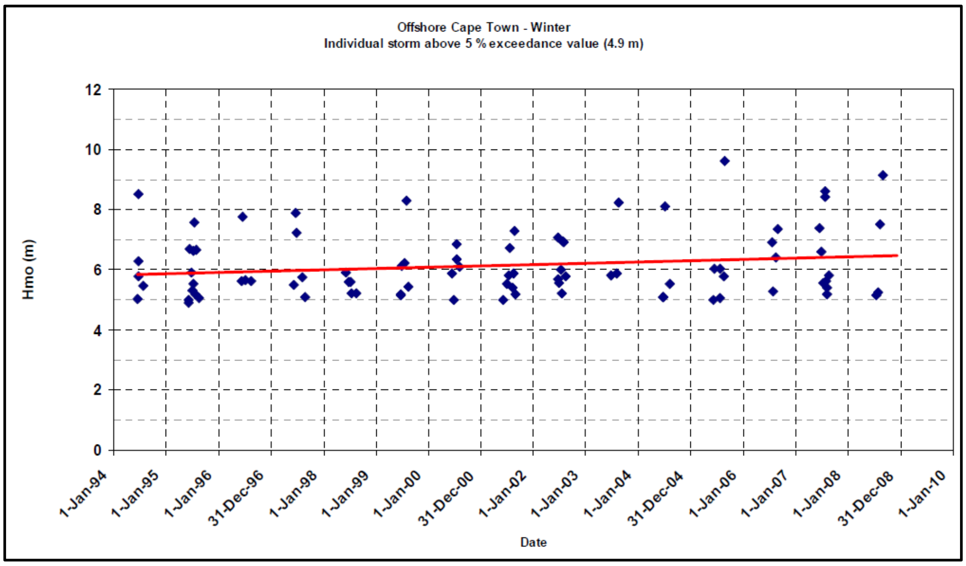


Figure 2-7: Peaks of individual storms over 14-year period – offshore Cape Town (Source: Rossouw, Theron 2012)

2.5.1 Sea level rise

The global mean sea level rose by 0.19 m over the period 1901 – 2010, with an average annual rate of 1.7 mm. The period 1993-2010 saw the annual rate increased to 3.2 mm/yr. Furthermore, it is projected that the global sea level will *likely* (66% - 100% likely) rise by 0.17 m – 0.38 m (95% confidence interval) by the period 2046-2065, and by 0.26 – 0.82 m (95% confidence interval) by the period 2081 – 2100 (values relative to 1986 – 2005 period) (Pachauri *et al.* 2015).

Local regional authorities have adopted sea level rise in long-term strategic planning for all new development (setback line methodology) and to assess the adequacy of existing coastal defence structures. Theron (2016) recommends the adoption of 1 m SLR by 2100 as a central best estimate to be adopted in setback line determination and coastal planning. Further values for the best central estimate SLR corresponding with the periods 2030 and 2050 were interpolated by Theron (2016) to be 0.15 m and 0.35 m respectively. These estimations were made from in depth research into climate change and SLR literature published after the cut-off date for submission to the AR5 IPCC (2013) report. These estimates of SLR will be adopted in this study when assessing extreme still water levels at the Great Brak estuary in Mossel Bay in Section 5.3.

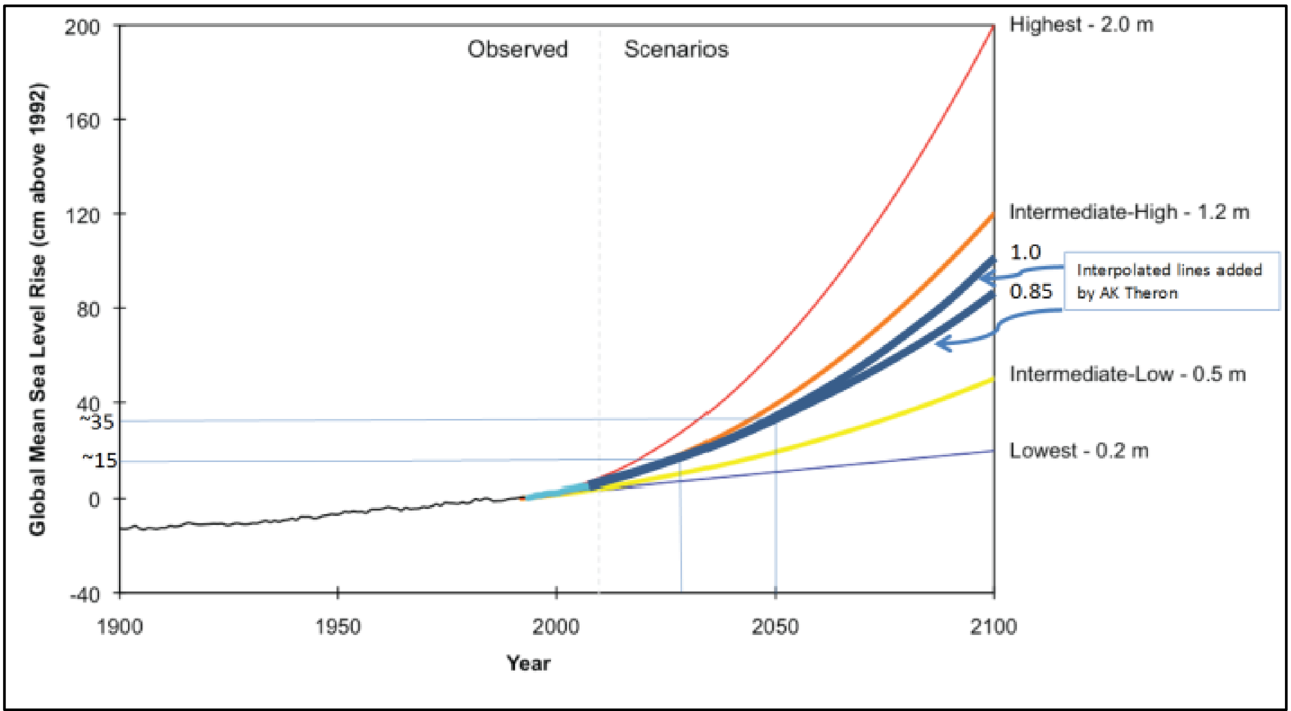


Figure 2-8: Global sea level rise prediction scenarios (Source: Theron 2016)

2.5.2 Overland precipitation trends

Change in overland precipitation patterns are also projected for the next century, due to climate change. Research into regional climate change impacts were undertaken in the Long-Term Adaption Scenarios (LTAS) Research Flagship Program. Various downscaled regional climate models were assessed and simulations determined the most significant impacts of climate change to be a change in drought and flood frequency and severity, sea level rise and changes to catchment sediment yield (LTAS., 2014).

See Figure 2-9 for the potential changes in annual run-off by 2040 – 2050 compared to the base scenario, for various secondary catchment areas. A likely reduction in streamflow were projected for the Western half of the country (catchments D to K) and a likely increase in flooding risk due to increases in run-off, for the Eastern half of the country. Secondary catchment K2 is relevant to this study. A negative change in the annual run-off is expected for this catchment.

The effect of climate change on the annual maximum daily rainfall (rainfall intensity) and the annual maximum cumulative daily run-off (annual flood peaks) were also assessed. The regional climate models indicate a large spatial variation in potential impacts across the country. No significant (>25%) increase in rainfall intensity was found for most of the country; however, for secondary catchment K2, an increase in the order of 5 – 25% increase in the 10-year rainfall intensity was predicted for the period 2045-2100. See Figure 2-10 for the results for the change in rainfall intensity of the five regionally downscaled climate models. The predicted change in the 10-year annual maximum daily run-off for the same period reflects the non-linear relationship between rainfall and run-off (LTAS., 2014). Some regions experience decreased flood risk and the risk for flooding increases significantly for others. All five models predict significant increases in flood risk in portions of the Western Cape. Most of the models predict an increase in the annual maximum daily run-off for catchment K2 between 50% - 100% (MPI, MIR, GF0), one model predicted an >100% increase (UKM) and another predicted a decrease (GF1). See Figure 2-11 for the results of the change in the 10-year maximum daily run-off for the period 2045 – 2100. The models also predict a surprising and worrying phenomena: the change in climate will

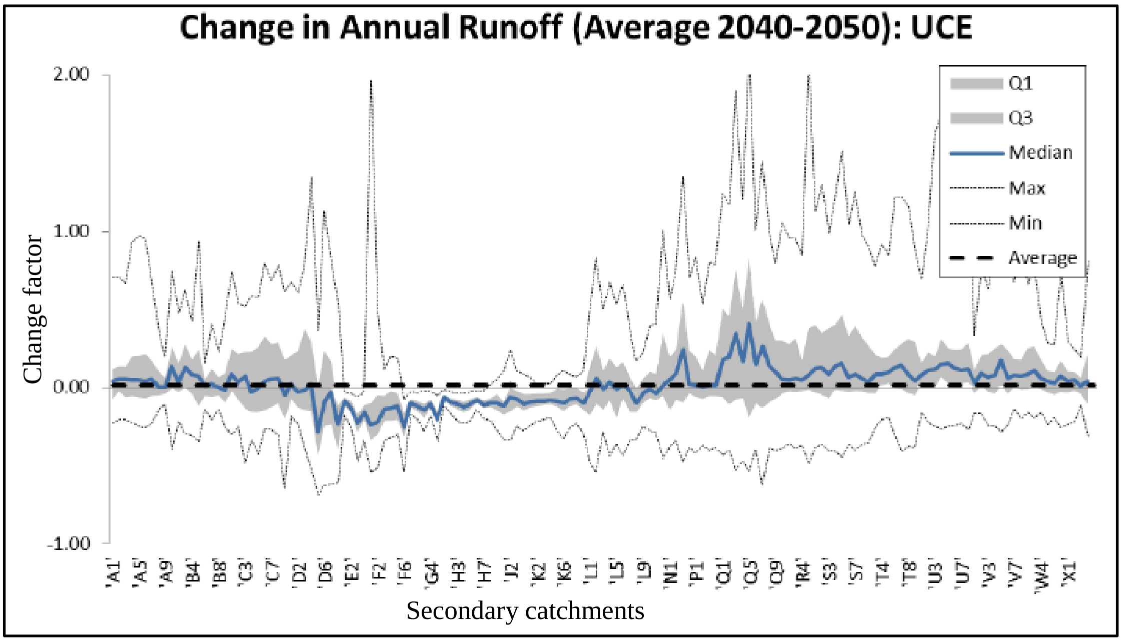


Figure 2-9: Potential changes to the average annual catchment run-off due to climate change, by the period 2040-2050 per the UCE scenario (Source: LTAS 2014)

impact the severity of the lower probability of occurrence extreme events (e.g. 100 - year storm) more than for the higher probability of occurrence events (e.g. 5 – year events) (in negative and positive changes) (LTAS., 2014).

The changes in catchment sediment yield were calculated using empirical sediment calculations derived for South Africa. Most dam catchments showed a positive increase in the mean annual sediment yield for the first half of the century. The effect on storage capacity, however, shows no significant potential

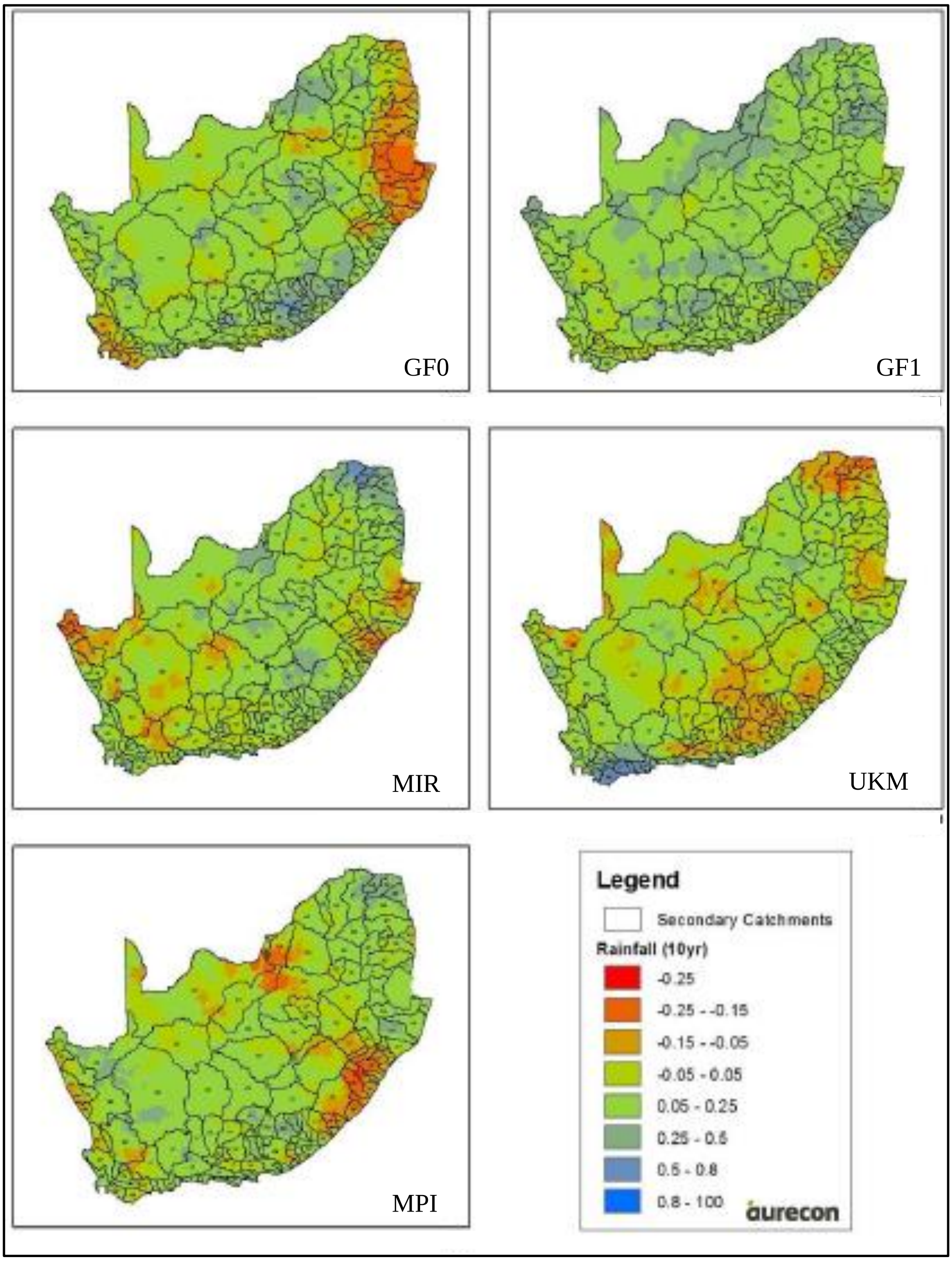


Figure 2-10: Possible maximum change to the 10-year annual maximum daily rainfall for the period 2045 – 2100 (Source: LTAS 2014)

loss of additional storage capacity (LTAS., 2014). This is attributed to the lesser impact that sediment yield has on areas with larger dams. Areas with small dams, however, may still experience significant sediment impacts.

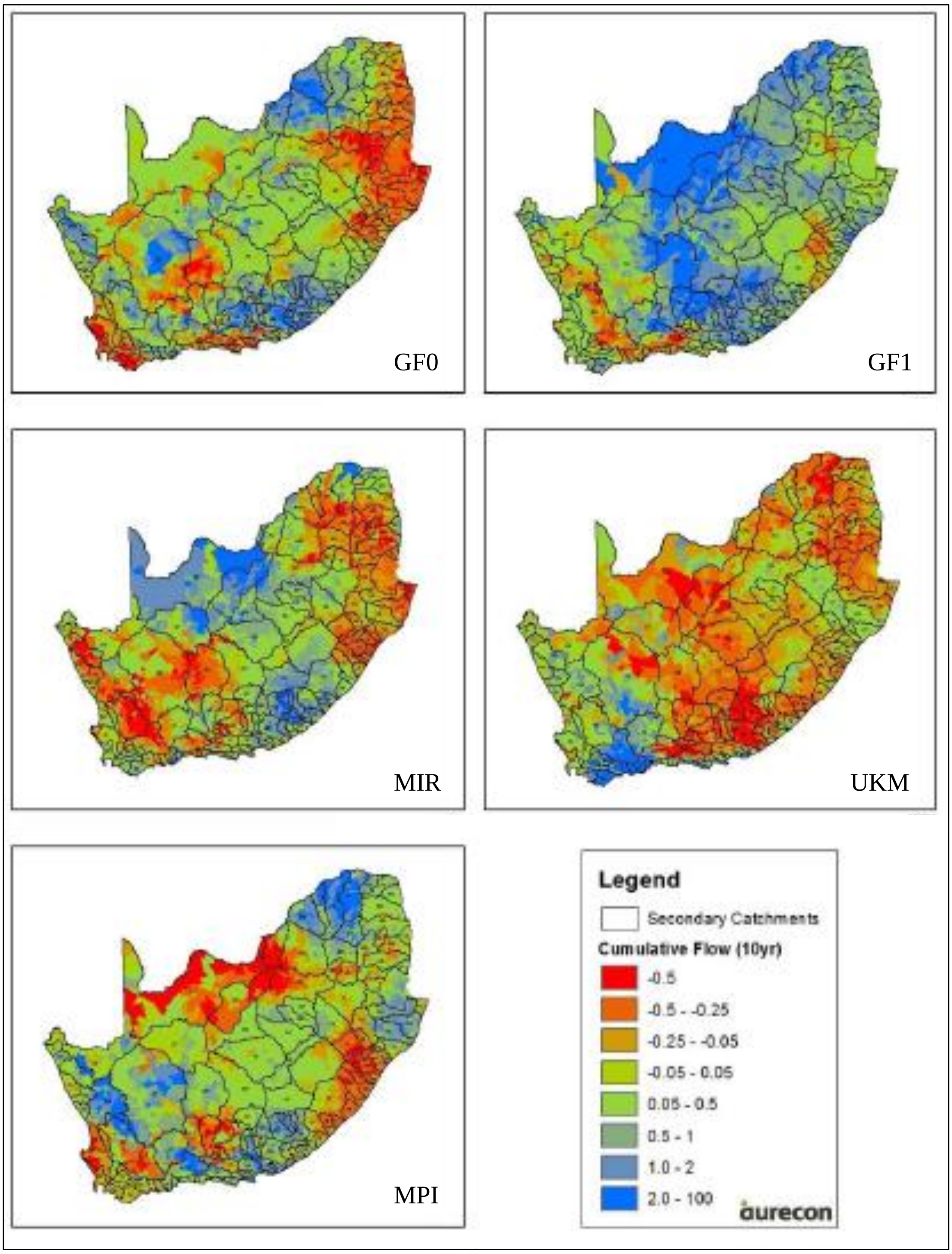


Figure 2-11: Possible maximum change to the 10-year annual maximum cumulative flow for the period 2045 – 2100 (Source: LTAS 2014)

2.5.3 Effect of climate change in estuarine water levels

From Section 2.5.1, it is apparent that a global mean sea level rise and a rise in frequency of extreme storm events can be expected for the next century – due to climate change induced global warming. Estuaries will feel the impact of sea level rise and an increase in storminess, seeing as it is vulnerable from flooding from both the ocean and the upstream catchment.

The biotic and abiotic features of estuaries are vulnerable to climate change. This section will focus on the hydrodynamic responses due to climate change and the impacts that the long-term changes can have on expected extreme water levels in the estuary. Du Pisani (2015) described what she considered to be the conceptual effects of climate change on estuaries in the context of the hydraulic responses. The long-term impacts of climate change effects were estimated to be related to changes in the sediment dynamics of estuaries due to either SLR or from changes in the run-off regime of the catchment.

Drivers of change in the estuarine environment are identified as storms, run-off and SLR. An increase in sea levels, storm frequency and intensity may cause significant changes in the sediment balance and the extent of tidal intrusion. Water levels inside estuaries are projected to rise, during both open and closed mouth conditions (Du Pisani. 2015). During open mouth conditions, the increase in sea level may negatively impact the flushing efficiency of the estuary during flood events due to the higher downstream water levels. The mouth berm could build up higher than normal and may migrate inlands due to an increase in ocean storm frequency and intensity. During closed mouth conditions, the water levels in the estuary during fluvial floods may rise to the height of the berm before it will start to overtop.

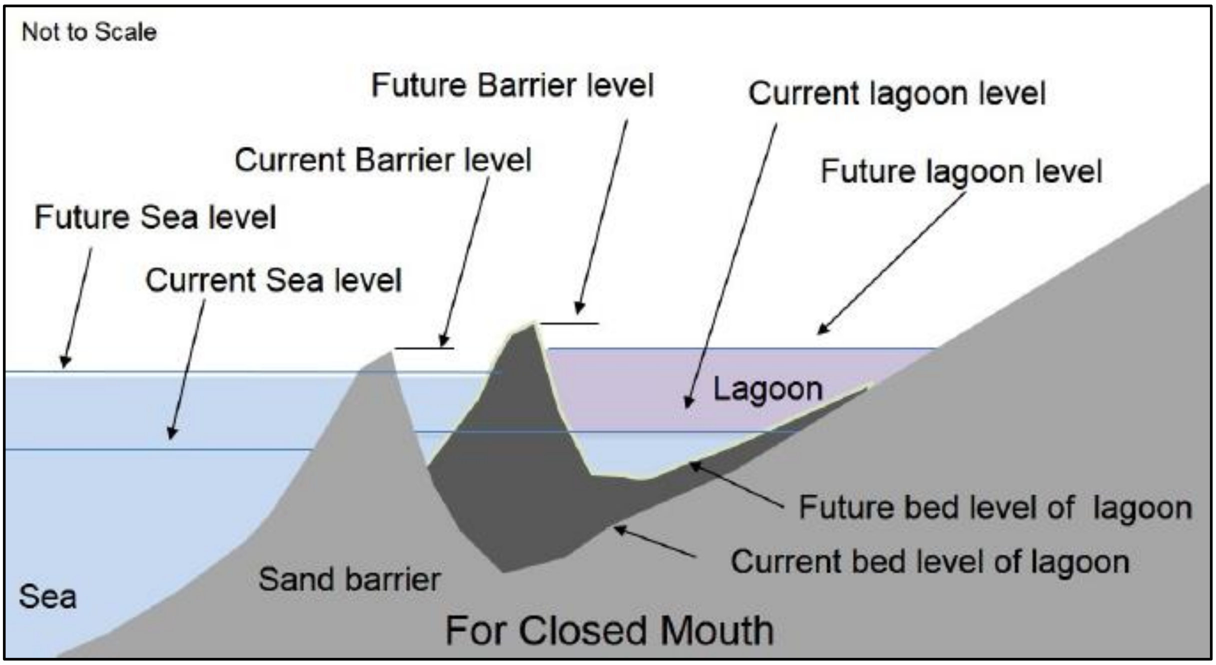


Figure 2-12: Schematisation of the estuary berm retreat due to climate change effects (Source: Du Pisani 2015)

See Figure 2-12 for a schematisation of the estuary berm retreat impact (as discussed above). It should be noted that these potential effects/impacts have not been verified in any manner.

2.6 Overland flooding

The nature of an estuary is such that it is susceptible to overland and coastal flooding. The probable overland flooding events for the catchment K20A will be discussed in this section. Over the years, the Great Brak estuary has been subjected to numerous studies investigating the hydrology of the catchment. The Wolwedans Dam has been identified as a hydraulic structure which imposes an attenuation and translation effect on passing floods, as the dam catchment is roughly 75% of the total catchment size (refer to Section 3.3 for more information of the relevant catchment area). This chapter will attempt to describe the past hydrological calculations for the catchment, the method of flood estimation that will be used in this study as well as the method of incorporating flood attenuation effects on extreme floods.

2.6.1 Hydrographs

This study will investigate the flood attenuation and translation effect of the Wolwedans Dam structure on floods from rainfall events in the catchment area above the dam structure. The flood routing calculation technique used in this study will be discussed in a subsequent section. This technique will need an inflow flood hydrograph as input. According to Roux and Rademeyer (2012), the Direct Run-off Hydrographic (DRH) method hydrographs should be used as input for flood routing calculations.

The DRH method incorporates a form of Moskingum routing method, assuming a uniformly distributed rainfall event over the catchment area, which is considered as a simple reservoir storage. The method is applicable to a catchment area between 20 km² to 20 000 km². The procedure to calculate effective storm rainfall is the same procedure used in the SUH method.

The Moskingum routing formula used to calculate the "outflow" hydrograph can be written as Equation 2-11. The driving mechanism for this model is rainfall duration (D), and the percentage of storm rainfall that fell in a percentage of duration is determined from a set of Hyetographs developed for different storm durations. The "outflow" hydrograph is calculated incrementally with a discreet time step between $0.1D \geq \Delta t \geq 0.05D$ (Alexander 2001).

$$Q_{out(2)} = C_0 Q_{in(2)} + C_1 Q_{in(1)} + C_2 Q_{out(1)} \qquad \textbf{2-11}$$

Where

$$Q_{in} = inflow\ (m^3/s)$$

$$Q_{out} = outflow\ (m^3/s)$$

$$C_0 = 1 - \frac{K}{\Delta t}(1 - C_2)$$

$$C_1 = \frac{K}{\Delta t}(1 - C_2) - C_2$$

$$C_2 = e^{-\frac{\Delta t}{K}}$$

And the K value applicable for South Africa (unpublished): $K = 0.6\, t_c$, where t_c is the time of concentration.

2.6.2 Flood routing

Flood routing is defined in the Drainage Manual (2013) as:

The influence of the storage characteristics between two spatial boundaries, on the discharge characteristics between the inlet and outlet flow rates when a specific hydrograph is assessed (SANRAL 2013).

A hydraulic structure, like a dam, imposes an attenuation and translation factor on a passing flood, due to the difference in the inflow and outflow hydrographs, which are influenced by the storage capacity of the structure and the spillway capacity. This relationship between the inflow, outflow hydrographs and the storage is important for the case at Groot Brak estuary, seeing as the Wolwedans Dam upstream of the estuary causes attenuation of floods in the catchment. The design flood determination of the catchment will be influenced by the dam structure, as the peak flow of the flood will decrease (attenuation) and the time the flood takes to achieve the peak flow will increase (translation). Flood attenuation is therefore the difference between the peak flow rates between the inflow and outflow hydrograph. Translation of the flood refers to the lag in time between the occurrence of peak flows for the inflow and outflow hydrographs (SANRAL 2013). See Figure 2-13 for a graphical representation of flood attenuation.

Flood routing through a level pool reservoir is a mathematical technique to determine an outflow hydrograph and is based on the continuity of mass principle (SANRAL 2013). For the application of this technique the following relationships for the hydraulic structure must be determined:

- Inflow hydrograph;
- Storage volume versus the water level in the reservoir;
- Storage volume versus outflow discharge;
- Stage versus outflow discharge;

❖ Initial values for storage, inflow and outflow

There are numerous methods to calculate a routing ability for a given hydraulic structure, i.e. Puls, iteration, Goodrich or by choosing an initial outflow value. Essentially, all methods aim to satisfy the continuity principle in the form of Equation 2-12. The level-pool reservoir (Puls) approach to determine the degree of attenuation is based on the continuity of mass principle which can be written as: the change in storage over a time period is equal to the rate of inflow minus the rate of outflow which is equal to the change in storage of the reservoir (See Equation 2-12). The solution method of the continuity principle described in SANRAL (2013) is in accordance with the method described in Hydraulic Structures (2007) and will be used to calculate the attenuation and translation effect of passing floods.

$$\Delta S = I\Delta t - O\Delta t \qquad \textbf{2-12}$$

Where:

ΔS = Change in storage volume (m^3)

I = average inflow (m^3/s)

O = average outflow (m^3/s)

Δt = time step (s)

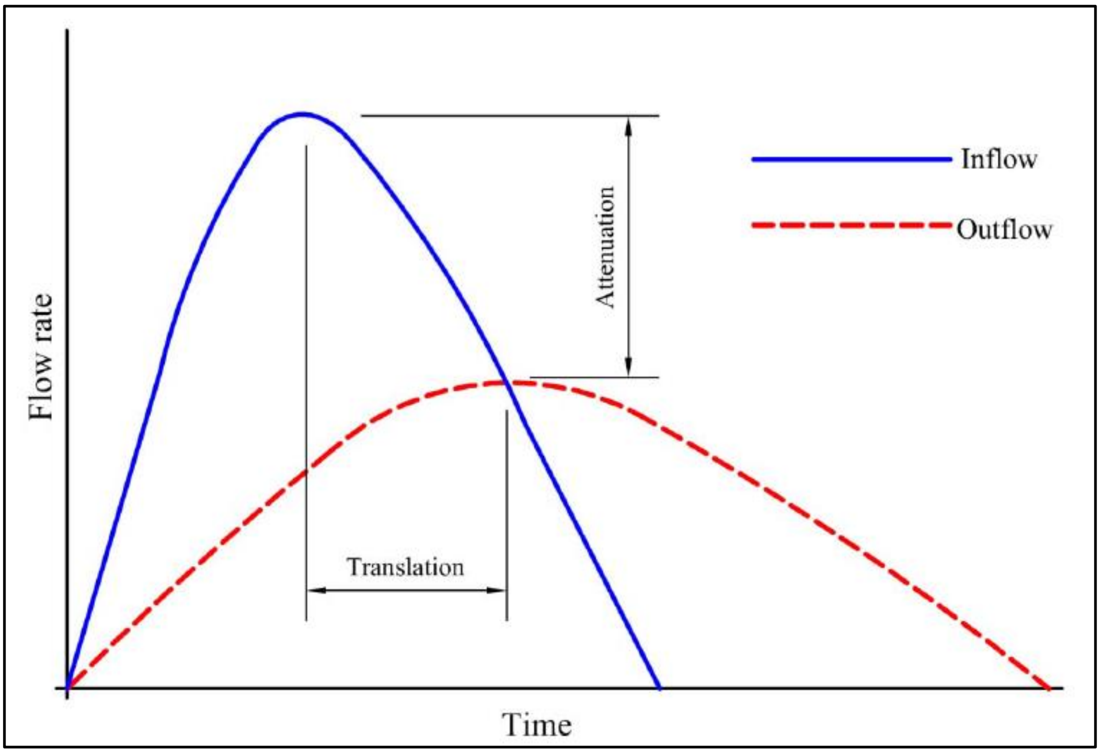


Figure 2-13: Typical inflow and outflow hydrographs of a hydraulic structure that cause flood attenuation (Source: SANRAL 2013)

Equation 2-12 can be rewritten to allow for approximation over a given time step, Δt. See Equation 2-13.

$$S_2 - S_1 = \frac{I_1 + I_2}{2}\Delta t - \frac{O_1 + O_2}{2}\Delta t \qquad \text{2-13}$$

The subscripts "1" and "2" denotes the start and end of the time, Δt, respectively. All parameters but S_2 and O_2, are known in Equation 2-13. By grouping the known and unknown parameters together, it follows that:

$$\frac{S_2}{\Delta t} + \frac{O_2}{2} = \left(\frac{S_1}{\Delta t} + \frac{O_1}{2}\right) + \frac{I_1 + I_2}{2} - O_1 \qquad \text{2-14}$$

A graphical relationship between the function $\frac{S_1}{\Delta t} + \frac{O_1}{2}$ and O_1 is then developed for a given time step, Δt, and denoted as the auxiliary function N (SANRAL 2013). This function will help to simplify the solution of the outflow time relationship. Equation 2-14 then becomes Equation 2-15:

$$N_2 = N_1 + \frac{I_1 + I_2}{2} - O_1 \qquad \text{2-15}$$

Where:

N = auxiliary function (m^3/s)

N_1 = $\frac{S_1}{\Delta t} + \frac{O_1}{2}$

N_2 = $\frac{S_2}{\Delta t} + \frac{O_2}{2}$

This principle of conservation of mass will be used to assess the Wolwedans Dam's attenuation effect given different starting volumes. The model setup and validation is discussed Appendix D.

2.6.3 Importance of large floods

Large flood events like the 50-year flood event (or larger) is important in estuary sediment dynamics. The large floods scour the estuary mouth and carries an important sediment load to the littoral system. The floods also ensure the flushing of potential previously deposited cohesive sediments that may consolidate in the mouth area (Beck, *et al* 2004). In areas where there are water retention structures upstream of the estuary, care should be taken to not influence of the frequency of large flood events by extensive attenuation.

2.7 Coastal flooding

Extreme coastal flooding events are caused by storm surge. Storm surge is a combination of atmospheric and climate phenomena which cause a rise in the inshore Still Water Level (SWL). The SWL at a location has a direct connection to the wave overtopping of structures and wave transmission that cause damage to structures and erosion (CIRIA 2007). The drivers of coastal flooding relevant to the South African coast are the effects of wind- and wave setup, high tides and inverse barometric setup (Theron. 2016). In future, the SWL will also be subject to sea level rise caused by climate change. See Figure 2-14 for a definition sketch of the various components of coastal flooding.

A combination of some of these phenomena results in extreme still water levels – which needs to be considered in the planning stage of any development near the shoreline. The methods for assessing the joint probability (i.e. Monte Carlo method) of occurrence for these phenomena require long periods of recorded water level data, to be statistically accurate. According to Theron (2016), such data is unavailable in South Africa. A precautionary approach is therefore used where plausible scenario combinations are assessed and is deemed a first level of approximation. The precautionary method entails the combination of the different components or by considering likely scenarios. The phenomena applicable to this study area will be further discussed in this section.

Further distinction will be made from static water level rising components, as discussed in preceding paragraphs, and more dynamic flooding levels caused by the wave – shore interaction. Wave run-up

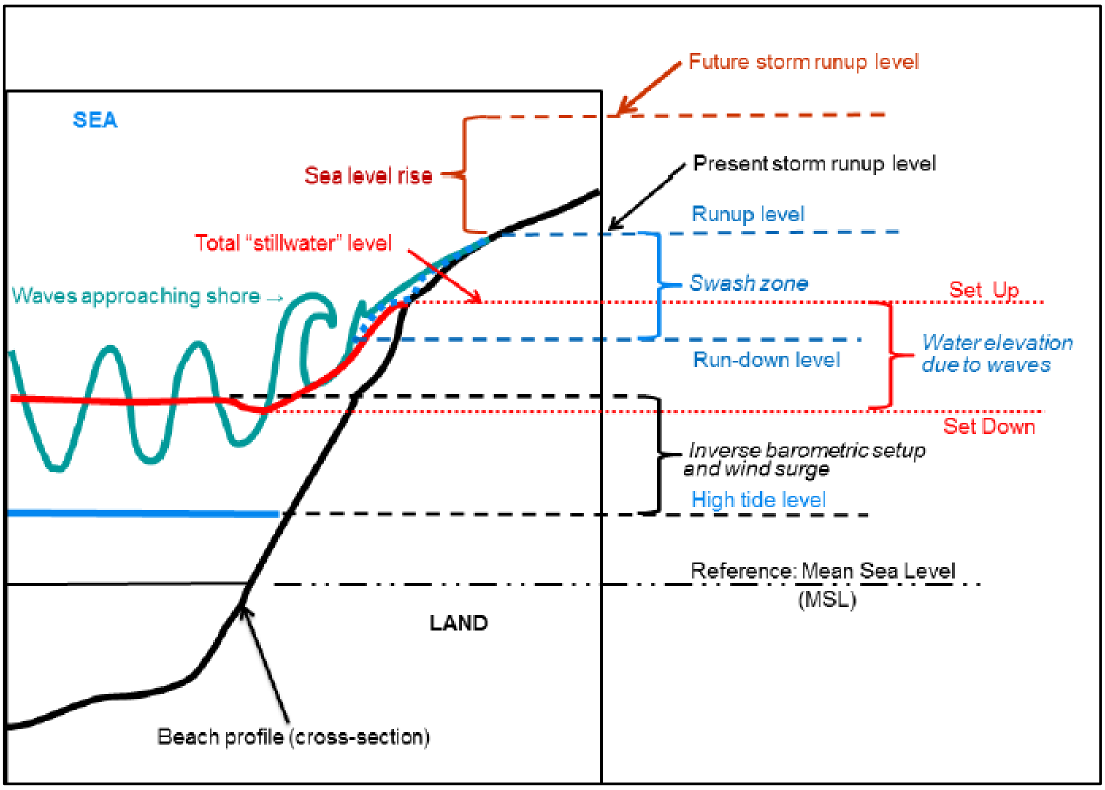


Figure 2-14: Various components leading to extreme inshore seawater levels (Source: Theron 2016)

and overtopping will also be defined in this section in order to assess the overtopping of the estuary berm and the transmission of waves into the estuary in a marine dominated environment, to simulate possible extreme events coinciding with open and close mouth states.

2.7.1 High tides

As discussed in Section 2.3.1, the drivers of tides are astronomical. The tides are generated by the fundamental attractions between the moon and sun. The tidal variation in South Africa is deemed semi-diurnal. Meaning that on any given day, the water level reaches high and low tide roughly twice daily with a wave period of 12.42 hours.

During full moon and new moon, the sun and the moon's gravitational forces work together to cause higher than normal tidal variation, i.e. spring tides, whereas during first and third quarter moons, the sun and moon's gravitational forces work against each other to cause smaller than normal tidal variation, i.e. neap tides. Due to the frequency of occurrence of these spring tides (every 14 days), the chance of a large storm event coinciding with a high spring tide is large enough to be a likely scenario to plan for.

The highest spring tides occur in spring and autumn each year, coinciding with the annual equinoxes. The highest equinoctial tides are close to the Highest Astronomical Tide, and occur every 4.4 years. The HAT is the highest tide that can be reached under average meteorological conditions in an 18.6-year nodal cycle (Theron. 2016).

The tidal variation around the South African coast is recorded at each major port around the country. The main component of these water level recordings is mainly tidal; however, wind effects and hydrostatic effects are also included in these recordings to an extent.

2.7.2 Wave and wind setup

Wave setup is the super-elevation of the nearshore water surface over the normal water surface elevation due to wave action (Theron. 2016) and should always be included in wave runup determination. This effect is amplified when the waves propagate into a semi-enclosed bay. Theron (2016) has highlighted that a rough estimate of wave setup may be taken as 10% to 23% of the breaker wave height. Various guidelines are stipulated in the literature to estimate the wave setup at a location, see Table 2-1. These authors estimate the wave setup as a function of various wave parameters, which includes the significant wave height at 10 m depth ($H_{S_{10}}$), the breaker wave height (H_b), the offshore wave height (H_{mo}), the "equivalent unrefracted" offshore wave height (H'_0) and the deep-water wave height (H_0) and length (L_0). Accurate location-specific wave setup should be determined by a detailed numerical model.

Table 2-1: Guidelines to estimate wave setup (Source: Adapted from Theron 2016)

Author	Wave setup estimation
Guza and Thornton (1982)	$0.17 \cdot H_{S_{10}}$
Priestly (2013)	$0.19 \cdot H_b$ with breaker index of 0.78
Callaghan *et al* (2008)	$0.23 \cdot H_b$ (maximum setup)
Karsten (2008)	$0.2 \cdot H_{mo}$
Dean and dalrymple (2001)	$0.17 \cdot H_{so}$
Goda (2000)	$(0.13\ to\ 0.15) \cdot H'_0\ for\ T_P < 12s$ $0.16 \cdot H'_0\ for\ T_P > 12s$
Stockdon *et al* (2006)	$0.016\sqrt{\frac{H_0}{L_0}}$

Wind setup is the additional elevation of the SWL caused by prolonged extreme wind conditions. As with wave setup, the wind setup is also amplified when the wind blows into a semi-enclosed bay. Wind setup is influenced by the wind speed and the sea slope. Around the South African coast, wind setup of more than 0.15 m is not unheard of. According to Theron (2016), the wind setup is included in the recorded sea water levels, to a degree, and overestimation of the combined setup (wind + wave) is possible if additional provision is made for wind setup.

2.7.3 Inverse barometric setup

The inverse barometric effect, also called hydrostatic setup (or set-down), is the rise or fall in inshore seawater levels because of extreme high and low atmospheric pressure over the ocean. An extreme high-pressure system over the ocean causes a drop in water levels, similarly an extreme low pressure system causes a rise in the SWL, which is the effect under discussion in this section. The effect of hydrostatic setup is seen on the east and southern coasts of South Africa due to the passing low-pressure cells and cut-off low systems.

The hydrostatic setup forms part of the recorded water levels at each port around the country. A rule of thumb is that a 1 hPa drop in pressure (from natural ocean conditions: 1 113 hPa) causes a 1 cm rise in SWL (Kamphuis 2012). Alternatively, the magnitude of this effect at a location can be calculated more accurately using a numerical model, simulating a storm by a developing a propagating pressure field.

2.7.4 Wave run-up

Wave run-up is a dominant component of coastal flooding around the South African coast (Theron. 2016). Wave run-up is defined as the extreme water level reached by a wave traveling up a beach slope, measured vertically. The run-up value is a function of wave direction, - height, - period, foreshore slope, wave breaker type, permeability of in- and nearshore profile and surf-zone width. The two percent exceedance run-up level, $R_{u2\%}$, is usually a significant design value for coastal structures and is used in the structure overtopping literature.

This run-up level can be calculated using empirical equations based on model test results or field data, otherwise a numerical model for wave/structure interactions can be used (CIRIA 2007). Theron (2016) evaluated a number of run-up models and compared it to field test data. The two run-up prediction models of (1) Nielsen and Hanslow (1991), (2) Mather *et al* (2011) was found to be the best applicable to the South African coast and for the specific conditions at Great Brak in Mossel Bay.

1. **Nielsen and Hanslow (1991)**

The empirical wave run-up model derived by Nielsen and Hanslow (1991) requires the beach face slope ($tan\ \alpha$), water level (WL), deep-water root mean squared wave height (H_{0rms}), deep-water wave length (L_0) and peak wave period (T_P). See Equation 2-16 for the condition $tan\ \alpha > 0.1$, and Equation 2-17 for the condition where $tan\ \alpha \leq 0.1$.

$$R_{u2\%} = WL + 1.98 \cdot \left(0.6 \tan\alpha \cdot \sqrt{\beta}\right) \tag{2-16}$$

$$R_{u2\%} = WL + 1.98 \cdot \left(0.05 \cdot \sqrt{\beta}\right) \tag{2-17}$$

Where:

$$\beta = \frac{H_{0rms}}{\sqrt{2}} \cdot L_0$$

And:

$$L_0 = \frac{g{T_P}^2}{2\pi}$$

The Nielsen and Hanslow (1991) model was adapted by Theron (2016) to better represent the case where $tan\ \alpha\ < 0.06$ and can be seen in Equation 2-18.

$$R_{u2\%} = WL + 1.98 \cdot \left(0.04 \cdot \sqrt{\beta}\right) \tag{2-18}$$

Theron (2016) concluded that the best run-up results were obtained for this model, by using the equivalent deep-water significant wave heights, as input parameter. The equivalent deep-water significant wave height is calculated by “reverse shoaling” the significant wave height determined at about 20 m depth to represent the theoretical deepsea conditions. An inverted shoaling coefficient is applied to the recorded wave heights to calculate the equivalent deep-water significant wave height.

2. **Mather *et al* (2011)**

The model of Mather *et al* (2011), for the 2% exceedance run-up level, $R_{u2\%}$, is a function of the WL, the deep-water significant wave height, H_0, the horizontal difference offshore to a specific depth, x_h (normally to 15 m depth) and a dimensionless coefficient, C. See Equation 2-19. Values for the dimensionless coefficient, C, are recommended to be taken as 7.5 for exposed coasts, 5 for large embayments and 4 for small embayments.

$$R_{u2\%} = WL + C \cdot H_0 \cdot \left(\frac{15}{X_{15}}\right)^{2/3} \tag{2-19}$$

2.7.5 Overtopping

Overtopping takes place when a run-up level for a certain wave is higher than the crest of the sea defence structure. A structure is usually designed for a certain accepted overtopping rate and volume, depending on the function that the structure has to perform. The EurOtop (2007) is a manual for engineers, focused on assessing the overtopping of an existing structure. The manual is based on field tests, mainly on European coasts, however, Theron (2016) states that overtopping around the South African coast can be adequately assessed by means of the EurOtop (2007) manual.

Overtopping is predicted in the EurOtop (2007) manual by quantifying overtopping discharges and volumes, the distribution of overtopping waves and the number of overtopping waves by using a probability of overtopping parameter. A structural response to a certain return period wave condition can therefore be assessed within a certain confidence level. The water level and the significant wave condition at the toe of the structure are used as input parameters for the methods described in the manual. The manual recommends including tidal and storm surge effects for extreme SWL and the significant storm wave in the overtopping assessment of a structure (EurOtop 2007).

Overtopping will be used in this study to assess the probable overtopping discharges and volume of the beach berm at the entrance of the Great Brak estuary under closed mouth condition, and for the sea defence structures around the Island (under investigation) when the mouth is open and significant wave energy can penetrate the mouth. The methods in the EurOtop (2007) will be used for this purpose. The freeboard, R_C, is an important parameter when assessing overtopping. The freeboard is defined as the difference between the crest height of the structure under consideration and the SWL. See Figure 2-15 for a definition sketch for various freeboard, R_C, scenarios. The beach berm is assumed to be an impermeable simple sloped structure.

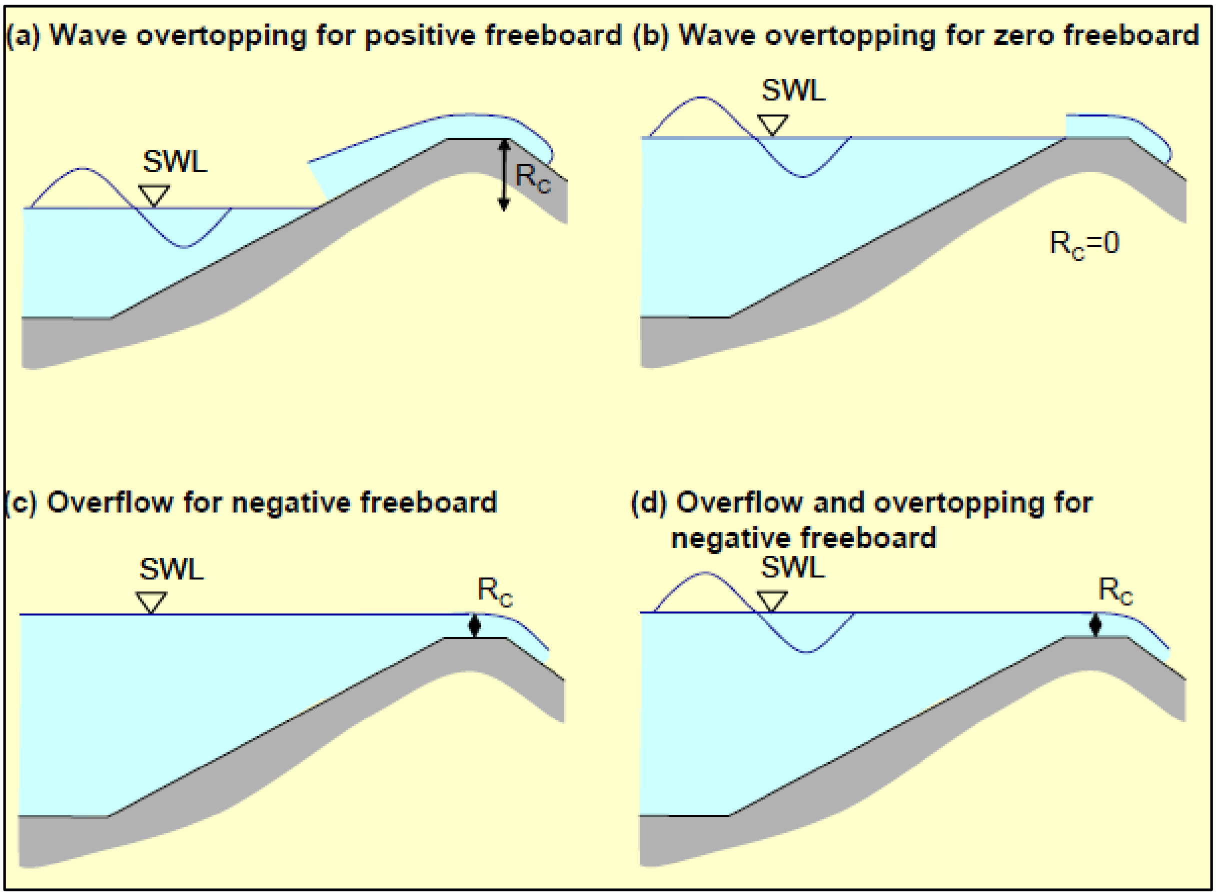


Figure 2-15: Wave overtopping definition sketch for positive, zero and negative freeboard (Source: EurOtop 2007)

2.7.5.1 Overtopping discharges

If the beach, or estuary mouth berm is assumed to be a structure with a smooth and simple foreshore slope, the overtopping discharge can be calculated using the formulas stipulated in EurOtop (2007).

When $R_C > 0$, and $\xi_{m-1,0} < 5$, the overtopping unit discharge (q in $m^3/s/m$) can be calculated using Equation 2-20.

$$q = \sqrt{g \cdot H_{mo}{}^3} \cdot \frac{0.067}{\sqrt{\tan\alpha}} \cdot \gamma_b \cdot \xi_{m-1,0} \cdot \exp\left(-4.3 \cdot \frac{R_C}{\xi_{m-1,0} \cdot H_{mo} \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v}\right) \qquad \text{2-20}$$

For the case when $R_C = 0$, the Equation 2-21 should be used to calculate the overtopping unit discharge, q.

$$q = \begin{cases} \sqrt{g \cdot H_{mo}{}^3} \cdot 0.0537 \cdot \xi_{m-1,0} \ \ for\, \xi_{m-1,0} < 2 \\ \sqrt{g \cdot H_{mo}{}^3} \cdot \left(0.136 - \frac{0.226}{\xi_{m-1,0}{}^3}\right) for\, \xi_{m-1,0} > 2 \end{cases} \qquad \text{2-21}$$

When $R_C > 0$, the overtopping component is less important than when the freeboard is negative. An overflow component also needs to be calculated and added to the overflow component. Equation 2-22 describes the unit discharge, q, for negative freeboard and for the case where $\xi_{m-1,0} < 2.0$.

$$q = q_{overflow} + q_{overtopping}$$

$$= 0.6 \cdot \sqrt{g \cdot |-R_C{}^3|} + 0.0537 \cdot \xi_{m-1,0} \cdot \sqrt{g \cdot H_{mo}{}^3} \qquad \text{2-22}$$

Where

q = mean discharge per meter ($m^3/s/m$)

R_C = freeboard (m)

H_{mo} = incident significant wave height at the toe of the structure (m)

$\tan\alpha$ = structure slope

$\xi_{m-1,0}$ = Irribaren number calculated as

$$\xi_{m-1,0} = \frac{\tan\alpha}{\sqrt{H_{mo}/L_{m-1,0}}}$$

With,

$$L_{m-1,0} = \frac{gT_{m-1,0}{}^2}{2\pi}$$

$$T_{m-1,0} = 1.1 \cdot T_P$$

2.7.5.2 Overtopping volume

The volume of water and the number of overtopping waves are not yet described by the overtopping unit discharge, q. To assess the maximum volume, V_{msx} (m^3/m), of water that will overtop the crest of a structure during a storm, the number of overtopping waves, storm duration and the overtopping unit discharge are considered (EurOtop 2007). Equation 2-23 describes the maximum volume of overtopping for a given storm.

$$V_{max} = a \cdot [\ln(N_{ov})]^{\frac{4}{3}} \qquad \text{2-23}$$

The scale factor, a, can be calculated with Equation 2-24.

$$a = 0.84 \cdot T_m \cdot \frac{q}{P_{ov}} \qquad \text{2-24}$$

The probability of overtopping per wave, P_{ov}, is a function of the freeboard, R_C, and the 2% exceedance run-up level relevant to the SWL. See Equation 2-25.

$$P_{ov} = exp\left[-\left(\sqrt{-\ln 0.02}\,\frac{R_C}{R_{u2\%}}\right)\right] \qquad \text{2-25}$$

Where the number of overtopping waves can be described as the product of the probability of overtopping, P_{ov}, and the total number of waves for the storm, N_w. See Equation 2-26. The total number of waves, N_w, are calculated using the storm duration and the mean period of the incident wave condition.

$$N_{ov} = P_{ov} \cdot N_w \qquad \text{2-26}$$

2.8 Conclusions from the literature

2.8.1 Estuary definition and classification

The literature review described an estuary and the various classifications of the fluvial-marine interface. South African estuaries differ from estuaries around the world and require a unique definition. The South African estuary experiences large variations in river run-off, from extreme flooding events to extreme low flow periods and establishes a connection to the ocean either permanently or periodically.

The variability of run-off causes the river inlet to experience either permanently open (POE) or temporary open and closed conditions (TOCE). The Great Brak estuary exhibits the characteristics of a TOCE river mouth (Whitfield classification). The southern Cape estuaries have been described as non-perched (Cooper 2001); however, during closed mouth conditions, the estuary may become perched. Breaching of the Great Brak estuary mouth with a volume release from the Wolwedans Dam creates a perched estuary situation in order to increase the flushing efficiency.

2.8.2 Estuary hydrodynamics

Dominant water flow regimes relevant to estuaries have been described to be driven by tidal currents and river run-off. In medium sized estuaries, like the Great Brak estuary, the tidal flow plays a large role in maintaining open mouth conditions, where the mouth may close during neap tide events. Larger estuaries (>100 ha) have large established tidal prisms, which helps to maintain open mouth conditions. The catchment size is not always linked to run-off, especially in an arid country like South Africa. However, POE are normally part of large catchments where there is full-year river flow. TOCE are generally part of smaller catchments with seasonal variation in run-off. The mouth of the estuary is dynamic and highly sensitive to flow modification of the river course. The frequency of open mouth conditions can be adversely affected by a reduction in run-off. The Great Brak estuary has experienced significant flow reduction due to the Wolwedans Dam and is dependent on ecological releases from the dam to open and maintain open mouth conditions.

Estuary mouth closing forces have been described as a combination of various factors with the dominant closing force being identified as wave action. Other factors like the beach slope, width of breaker zone, marine and fluvial sediment availability, sediment grain size and berm height are all factors that influence the closing of the mouth. Ranashinge *et al* (1999) described two closing mechanisms that are applicable to small estuaries in meso-tidal and wave dominated environments. These closing mechanisms are driven by longshore and cross-shore sediment transportation processes, which is driven by seasonal wave conditions. Some geologic (rocky headlands) and marine structures (reef systems in the surf-zone in front of tidal inlets) have been described to facilitate open mouth conditions.

To calculate the extreme event flood lines of the estuary, in-depth hydrodynamic numerical modelling is the best method to employ. It is important to address the dynamic nature of the inlet geometry and open and closed mouth conditions. When an estuary is closed, it functions essentially like a dam, and is governed by fundamental laws, like the conservation of mass and momentum. Expressions for an ideal estuary basin model have been described, and will be used in this study in an attempt to calculate first order estimations of possible water levels in the estuary. No analytical approximations have been found that can describe the scouring of the estuary mouth.

2.8.3 Climate change

The estuarine environment is vulnerable to climate change effects in ocean sea levels and from shifting overland precipitation patterns. The main components of climate change relevant to estuarine environment were investigated and found to be the combined effects of sea level rise, changes to the annual maximum daily rainfall (rainfall intensity), changes to the annual maximum daily run-off (storm peak flows) and to the annual run-off. Changes in sediment yield from the catchment were found to be a secondary factor of climate change, as the result of overland precipitation trends, and applies to the estuarine environment.

SLR is the main marine component of climate change that will have a direct impact on estuaries. Theron (2016) recommended values for the central estimate of SLR to be used in coastal planning practices, and will be adopted for the purposes of this study. See Table 2-2 for these values. From recent regional wave climate research by Theron and Rossouw (2012), the significant wave height during storms seem to be on the rise as well. This was deemed to be an indicator of an increase in storminess in the ocean, influencing the storm severity.

Table 2-2: Adopted climate change values for this study

Period	Value
By year 2030	+0.15 m
By year 2050	+0.35 m
By year 2100	+1.0 m

From the results of the downscaled regional climate model simulations by LTAS (2014), catchment specific estimations can be drawn. The overall annual run-off for the catchment K2 is predicted to be reduced by approximately <15% by the period 2040 – 2050. However, the rainfall intensity and the resultant flood peaks is said to increase by the year 2100, for this catchment. Rainfall intensity is predicted to increase by <25% and the flood peaks will increase much more significantly, with majority of the models predicting an increase between the range 50%-100%. This means that the catchment will receive less river run-off annually, but the extreme storm intensity and frequency will increase, causing more frequent and more severe storms. The catchment sediment yield for small catchments will increase over the first half of the century, which may have an impact on catchment K2.

The impact of SLR on the estuary mouth berm was found to be the major direct negative influence of climate change. The estuary berm is predicted to migrate inland, as the sea level rise and size of storm waves increase. This may lead to a dangerous situation for the Great Brak estuary, as the higher berm level and shortened estuary may cause extreme water levels to be reached more frequently.

2.8.4 Extreme flood events

The estuarine environment will experience flooding from extreme ocean events and from extreme overland rainfall events. The catchment is very small and has a quick response time. Flash flooding from rain in the upper catchment can reach the lower estuary basin very quick if the dam is full. Large waves can overtop the closed mouth berm or will be able to penetrate the estuary inlet and reach the Island, if the water level is high enough to allow waves to propagate into the inlet.

The Wolwedans Dam and the estuary mouth berm has proven to be the most important hydraulic structures upstream and downstream of the Island that plays a significant role in providing flood attenuation. For an extreme fluvial flood, the best-case scenario is a less than full dam, an open mouth condition and a low tidal level. For an extreme marine event, a closed mouth will protect the Island from direct wave attack, but if the mouth berm is not high enough, overtopping of the berm from the large waves may cause extreme backflooding in the lower estuary basin. A flood defence option for the Island will need to address large waves generated by storms as well as large fluvial floods.

To estimate the overland extreme floods, the Direct Run-off Hydrograph (DRH) method will be used. The routing of the Wolwedans Dam will be addressed through an adaption of the level-pool routing technique and a dam basin model. The joint probability of occurrence of ocean storm events will be addressed by superimposing plausible scenario components on the tidal level to estimate the extreme still water levels, these components will incorporate storm surge as a combination of wave-, and wind- and inverse barometric setup. The extreme wave conditions for Mossel Bay have been determined by an extreme value analysis by Clarke (2016), and will be used in this study. Wave run-up will be calculated for the area at Groot Brak by means of two models, recommended by Theron (2016). These models are the Nielsen and Hanslow (1991) model and the Mather *et al* (2011) model. The overtopping discharge and the probable overtopping volume will be calculated with the methods stipulated in the EurOtop manual (2007).

3. Situation assessment: Great Brak Estuary

This section will investigate the situation of the Great Brak estuary in depth. A short history of the town, the Estuary Management plan, catchment characteristics, historic flood events and the mouth conditions and manipulations of the estuary mouth will be discussed. Hydraulic modelling efforts for the Great Brak estuary done in recent years will also be investigated.

3.1 History

The Great Brak River was first described by pioneering trek farmers in 1730. After which, in 1745, the river became the eastern boundary of the Cape Colony. Francois le Valliant described the area in 1782 as part of the "most beautiful country in the universe" after crossing the river, climbing the mountain (Great Brak Heights) and admiring the view of the Outeniqua mountain chain as well as the surrounding forests, plains and hills (Franklin 1975).

A causeway was built across the river in the 1840's and 1850's when roads were improved in the area after the north-eastward expansion of the Cape Colony. The causeway consisted of thirteen stone piers with 6 m openings in between, spanned with timber. In 1852 the causeway became a toll bridge (Franklin 1975).

The town of Great Brak was founded in 1859 by Charles Searle, a British settler. The Searle family first oversaw the tollgate at the causeway. This was the start of the long relationship between Great Brak and the Searle family. The town expanded around the causeway as accommodation for travellers and a post office in addition to shops were established (Franklin 1975). The town continued to expand during the 19th century. A 2.3 km long diversion channel was built in 1874 for irrigation of crops along the river and for operation of a wool washery and to power a corn mill. Willie Searle, son of Charles Searle, in 1902, suggested a unique water scheme that would provide electrical power to the factories in Great Brak.

In other developments in the town, "The Island", the land at the mouth of the Great Brak river, was in the spotlight in the 1920s and 1930s. In 1926, it was decided to develop the island in order to "popularise the sea-front" (Franklin 1975). The first bridge constructed to connect the "island" to the mainland consisted of wood, attached to drums floating in the water, to allow the rising and falling of the tide. After the reclamation of the swamp-part and the development of all the available land on the "island", in the 1950s, there were reportedly 80 plots of land.

Later in the 20th century, following increasing water demands from the town and the surroundings, a decision was made to build a dam in the river catchment, the Wolwedans dam. The dam was completed in 1990, and has a gross storage capacity of 25 million m^3. This saw the establishment of the Great Brak

River Environmental Committee (GEC), whose goal was to oversee the investigations into the possible environmental impacts the dam would have on the estuary and to draw up a management plan.

3.2 Estuary Management Plan

After the decision was made to build a dam in the Great Brak river, there were obvious concerns about the health of the estuary. The dam would undoubtedly reduce the inflow of fresh water into the estuary. The GEC was appointed by the DWA and represented members of the concerned public, national authorities and local authorities.

The duty of the GEC was to see to the investigation of the potential impacts that the dam can have on the estuary and to establish a management plan for the maintenance of the estuary. The goal was to cause as little disturbance to the estuarine environment as possible (Barwell, *et al*, n.d.).

The CSIR was approached by the GEC to carry out the Environmental Impact Assessment and to draw up a management plan to be reviewed by the GEC. An integrated environmental management (IEM) approach had to be followed as stipulated by the DWA. The CSIR made predictions of possible changes to the estuary and set up a management plan, which included recommendations for monitoring programmes to ensure a healthy estuary and a dam release policy. The predictions on possible changes were made for mouth stability, socio-economics, estuarine ecology and the water quality. It was these secondary effects that were recommended to be monitored to assess the health of the estuary.

The EMP contained three main objectives, which would help achieve the goal. These objectives were as follows:

- Maintain the ecosystem as close as possible to natural state
- The aesthetic quality of the estuary and the tidal influence in the lower reaches of the estuary had to be maintained
- Maintain the potential recreational value of the area, especially during peak season

To achieve the objectives, the EMP had to be directed to achieve open mouth conditions for as long and frequent as possible (Barwell, *et al*, n.d.). The mouth state can be artificially altered by means of controlled water releases from the dam or by mechanical opening using a bulldozer, or a combination of the two methods.

The EMP incorporated a weighted scoring system and a checklist to continuously monitor the key parameters (water level, water quality, socio-economics and estuarine ecology) that are subject to change in the estuary. A rule-based decision support system is used in conjunction with the weighted score to trigger an action route to handle a variety of scenarios. See Appendix A for the estuary checklists used in the monitoring of key parameters (See Table A-1 and Table A-2).

3.2.1 Scenarios of water availability

Three scenarios were identified that would require the assistance of the maintenance body. The scenarios are three periods referring to the rainfall in the catchment area. The ideal scenario is high rainfall/run-off periods where the dam operates at full supply capacity (FSC) where the estuary can be managed like pre-dam conditions.

The predicted mode situation was identified as the periods where the dam is operating at 40% - 100%. The EMP covered the ecological, socio-economic and water quality requirements of the estuary within these periods. Following suggestion from the CSIR, the mouth of the estuary was to be opened no less than three times per year, in November, February and in June. These openings were to be done per the guidelines in the EMP plan to be kept open as long as possible, preferably for one month at a time. In these periods, coordinated releases from the dam was required to aid in breaching the mouth. A minimum release of 2×10^6 m^3 of water was to be made available to the estuary (Barwell, *et al*, n.d.).

During the least favourable scenario, in drought periods where the dam operates at less than 70% capacity, the ecological needs of the estuary then enjoys priority as the socio-economic needs are noted but ignored. The estuary will need to be breached at least once a year and be kept open for a month. It was decided that the end of November would be the best time for this opening. The water quality in the estuary deteriorates during these periods and mechanical mouth openings without the help of controlled releases was recommended if necessary. At least 1×10^6 m^3 of water was made available for the estuaries' needs during these periods.

The scenarios were used to establish a water release policy (Section 3.2.2) for the Wolwedans Dam. The water release policy was adopted into the EMP and supplied guidelines for monthly releases in relationship with the state of dam levels.

3.2.2 Dam water-release policy

The water made available for the needs of the estuary were to be managed in the most optimal way. The EMP stipulates a methodology to use this water when needed. The water-release policy, adopted in 2004, supplies a guideline as to the monthly relationship of releases versus dam water level for the purpose of keeping the mouth open (Council for Scientific and Industrial Research 2004).

Table 3-1: Water-release volume (in million m^3) in relationship to the volume of water (in %) in the Wolwedans Dam (CSIR 2004)

% in dam	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Total
> 90%	0.6	0.7	0.7	0.5	0.5	0.5	0.4			0.5		0.6	5.0
80 – 90	0.6	0.7	0.7	0.5	0.5	0.3							3.3
70 -80		0.7	0.7	0.5	0.3								2.2
<70%			0.7	0.3									1.0

From Table 3-1 can be seen that a minimum of 1 million m^3 will be made available for the ecological requirements of the estuary. Also, the recommended releases are centred around the spring and summer periods, to ensure open mouth conditions for the holiday period and for ecological reasons.

Water releases to assist in the management of the estuary mouth are to be done for the following reasons (Council for Scientific and Industrial Research 2004) :

- **Breaching:** When the water level in the estuary is to be increased to the desired breaching height. Normally between $0.5 \times 10^6\ m^3$ to $0.7 \times 10^6\ m^3$ is needed.
- **Maintain open mouth (1):** During neap tides in the summer and spring periods, between 0.1 $\times 10^6\ m^3$ and $0.2 \times 10^6\ m^3$ is needed to be released at a constant low flow of 0.5 m^3/s to keep the mouth open. The success rate is largely dependent on the wave conditions and is reportedly 50 – 70%.
- **Maintain open mouth (2):** To flush the mouth open, within one day of closing, a volume of $0.15 \times 10^6\ m^3$ to $0.4 \times 10^6\ m^3$ is needed for this. A success rate of 70% is reported for this method.

The amount of water to be released to assist in breaching is calculated by the authorities based on an estuary water level - volume relationship derived by engineer Piet Huizinga (Huizinga. 2017 pers. comm.). The current water level in the estuary can be used to calculate the volume of water needed to be released by the dam for a breaching activity.

3.2.3 Artificial breaching

The EMP made provision for recommendations on the artificial breaching of the estuary mouth. The breachings were to be done at the right time and water level to ensure the required scour efficiency that will allow for the mouth to stay open for longer periods.

Ideally the breaching had to occur four days after neap tide and about two hours before low tide. The excavated trench must be as deep and wide as possible and located where full advantage could be taken from the natural ebb and flow channels. The initial EMP recommended that the water level in the lower estuary be raised to +1.62 m MSL before breaching and that a steady release from the dam at a rate of 5 – 10 m^3/s during breaching be made to minimise seepage losses and prevent pollutants at the bottom of the estuary from being mobilised (Barwell, *et al*, n.d.).

The desired water level for breaching increased over the years from +1.62 m MSL to +2.0 m MSL. The maximum average outflow increased from < 10 m^3/s to 66 m^3/s due to the increase in average breaching height. It was deemed favourable to breach at the highest possible water level by the CSIR (2011), as the scour efficiency is better. The sediment of the berm can be successfully flushed out to ensure longer open mouth conditions.

During natural conditions, the berm will be breached when the water level in the estuary reach a level higher than the berm. This can cause a dangerous situation for the low-lying properties adjacent to the estuary, as the berm height can exceed +3.5 m MSL. The lowest property in the island is at +2.267 m MSL (Council for Scientific and Industrial Research 1990).

The height and width of the berm can play a significant role in the scour efficiency and the height of water needed to flush the mouth. Following long periods of closure, as demonstrated during the drought conditions in 2009, the estuary mouth was closed from July 2009 to February 2011 and the berm built up much wider than normal during the closure (Council for Scientific and Industrial Research 2011). The mouth was breached again in February 2011 but because the sheer size of the berm, the flushing of sediment was not as successful and the water level in the estuary only decreased from +2.02 m MSL to +0.969 m MSL and the maximum outflow only reached 11.44 m^3/s.

During neap tide, the event of mouth closure can be estimated from the low water level. If the estuary drains to a water level of 0.5 m MSL, the mouth is not likely to close. When the low water level in the estuary reaches 0.6 m MSL, during neap tide, the mouth might start closing and at 0.7m MSL, the mouth is very likely to close (Council for Scientific and Industrial Research 2004).

3.2.4 Emergency protocol

In the estuary monitoring report of 2011, the CSIR recommended that considerably more water should be made available for the estuary mouth management, specifically to perform precautionary breaching in the event of emergency conditions. Aspects like dam level, estuary water level and berm height were recommended to be monitored continuously to identify emergency conditions that would warrant a precautionary breach.

The catchment area, K20A, is only 164 km^2, thus the certainty of the prediction tools should be very high to ensure the rainfall will end up in the catchment area. The nature of storm paths is very dynamic and depend on various atmospheric conditions. There has been a case where 80-mm rainfall event was predicted for the catchment – but only 19 mm fell (24/25 May 2011). Two weeks later, a 79.2 mm rainfall event caused extensive flooding in the Great Brak estuary as a record water level in the lower estuary basin was achieved during the flood. Therefore, the criteria to identify emergency conditions need to be adequately defined to not waste any water.

Recently (since 2004) the Authorities do take a proactive stance in preventing extreme water levels to be reached in the estuary. The EMP was amended to specify which key elements to monitor on a continuous basis, in terms of identifying emergency conditions. These elements to be monitored are listed as follows (Kriel and Van Wyk. 2017 pers. comm.):

- Water level in the Wolwedans Dam, and its rate of increase;

- The actual and expected rainfall in the catchment (SAWS);
- Severe weather alert framework (Watch/warning issued by SAWS);
- Water level in the estuary and its rate of increase;
- Actual height and width of mouth sand berm;
- The actual and predicted wave conditions;
- The availability of equipment to breach the mouth on short notice, if necessary.

The mobilisation of necessary equipment for breaching, on short notice is the responsibility of the Municipality, if needed. See Figure 3-1 for an example of a breaching event by means of a bulldozer. The decision flow-chart governing the breaching in the event of an emergency can be seen in Figure A-5, Appendix A.

Summaries of breaching events from 2012-2017 were obtained from Mossel Bay Municipality (Kriel and Van Wyk. 2017 pers. comm.) and subsequently used to update Table B-1 to include all recorded breaching events until May 2017. The summaries show the cooperation of the Great Brak Environmental Committee and the Municipality of Mossel Bay in monitoring the abovementioned elements.

When a warning was issued by the South African Weather Service (SAWS) or a large rainfall or ocean storm event was forecasted, the Mossel Bay Municipality would send a message to All Interested and Affected Parties to inform them of the current dam level, estuary water level, estuary mouth status and mouth berm height as assessment of current conditions. The predicted and actual rainfall from SAWS reports and wave conditions from the CSIR WaveNet model are also attached to these update reports. The summaries show that if emergency conditions prevail, the breaching equipment (Bulldozer) was mobilised and arrangements were made for an assistive release from the Wolwedans Dam, after which breaching commenced.

The emergency conditions for which a breach is sanctioned was identified from the summaries of the 5-year period and found to be when combination of a full dam (> 98%), closed mouth, elevated berm height (>+2.5 m MSL) and elevated estuary water level (>+1.6 m MSL) all coincide with a predicted large rainfall event in the catchment, or when large waves are predicted.

The summaries also show an increase in flushing events, in attempts to keep the estuary mouth open after it closes from a neap tide cycle. It can be seen from Table 3-9 that the mouth of the estuary was approximately open for 53.7 % in the period from 1988 – 2011. In recent years (2012-2015), the estuary mouth is reportedly open 75.5 % of the time, which can be attributed to the increase in flush event frequency.

Figure 3-1: Emergency breach of the estuary mouth – 08/06/2011 (Source: Huizinga 2017)

3.3 Catchment characteristics

The most important features of the Great Brak river catchment will be described in this section. The characteristics of the dam, existing monitoring devices situated in the catchment and the hydrological information available for the catchment are all relevant to the larger picture of the study area.

3.3.1 Catchment size and precipitation

The quaternary catchment, K20A, is draining into the ocean via the Great Brak river. The catchment information from already existing literature can be seen summarised in Table 3-2. Final values for the catchment characteristics will be discussed in subsequent chapters.

Table 3-2: Physical characteristics of quaternary catchment area K20A

Characteristic	Value	Reference
Quaternary catchment	K20A	
Catchment size	164.34 km^2	DWA google earth layers (2017)
	156.6 km^2	(Pieterse 2014)
	192 km^2	(Midgley, Pitman 1969)
Longest watercourse	29.3 km	(Council for Scientific and Industrial Research 1987)
	29.9 km	DWA google earth layers (2017)
	31.7 km	(Pieterse 2014)
	31.5 km	(Council for Scientific and Industrial Research 1987)
Mean slope	5%	(Pieterse 2014)
	4%	(Council for Scientific and Industrial Research 1987)
Veld type	Zone 2	(Pieterse 2014)
MAP	775 mm	(Council for Scientific and Industrial Research 1987)
	750 mm	(Pieterse 2014)
	730 mm	(Roux, Rademeyer 2012)
Virgin MAR	37 million m^3	(Department of Water Affairs and Forestry 1991)
	36.79 million m^3	(Department of Water Affairs and Forestry (DWAF) 2008)

3.3.2 The Wolwedans dam

The Wolwedans dam is a hydraulic structure, located 8 km upstream of the estuary mouth that has a significant impact on the estuary via flow reduction. The dam wall was completed in 1989 and started filling in May 1990. With an average annual rainfall of 1 200 mm, in the mountains in the catchment, and 500 mm along the Great Brak river, the average run-off was 37 million m^3 per annum. The Wolwedans dam was designed to impound 65% of the catchment run-off (Department of Water Affairs and Forestry 1991).

The dam was built within the allocated time and budget of R48 million (February 1991 prices). Another R38 million was spent on pipelines, a pump station and reservoirs in order to complete the reticulation system. The main purpose of the dam was to establish a supply of water for the Mossgas plant. The plant focusses on the conversion of natural gas, found beneath the sea bed, into usable fuel. It was foreseen that the town of Mossel Bay would also require water by 1995 or shortly thereafter.

Figure 3-2: The Wolwedans dam busy overflowing (Source: Google Images 2017)

The dam is an arch – gravity, rollcrete structure. The dam wall is 70 m high with a crest length of 270 m. The spillway is an uncontrolled ogee-type spillway, with a maximum overflow of 1 920 m^3/s. The gross storage of the dam is 25 million m^3 and has a 110-ha surface area at full supply level. The dam was designed to yield an estimate of 10.2 million m^3 per annum, with 5.6 million m^3 to be allocated to Mosgas, 1 million m^3 for downstream ecological needs and the rest will be used for urban water supply in the Mossel Bay region (Department of Water Affairs and Forestry 1991). See Table 3-3 for a summary of the important information of the Wolwedans dam and Figure 3-2 for a visual of the dam wall.

Table 3-3: Parameters of the Wolwedans dam (Adapted from: Department of Water Affairs and Forestry 1991)

Parameter	Value
Year of completion	1989
Purpose	Water for industrial, domestic and ecological use
River	Great brak river
Type of dam	Rollcrete arch-gravity
Gross storage capacity	25 million m^3
Spillway type	Uncontrolled ogee-type
Design spillway capacity (RDD)	960 m^3/s
Maximum spillway capacity (SED)	1920 m^3/s – obtained from stage-discharge curve
Wall height above lowest foundation	70 m
Crest length	270 m
Arch radius (extrados)	135 m
Catchment size (gauging station: k2r001)	123 km^2
Owner, design and construction	Department of water affairs and forestry

3.3.3 Gauging Stations

In the quaternary catchment K20A, there are some river flow meters, water level, reservoir and rainfall stations. These data capturing instruments will be used to assess the hydrological and hydrodynamic situation in of the catchment. The relevant flow gauging stations downstream of the Wolwedans dam can be seen in Figure 3-3. Information regarding the gauging stations in K20A are listed in Table 3-4. K2R002 measures the monthly volume of water flowing over the spillway, K2H006 measures the daily flow rates just downstream of the dam wall and is rated to 68 m^3/s, K2H002 measures daily flow further downstream of the dam and is rated to 55.5 m^3/s and K2T004 measures water levels in the lower reaches of the estuary, mainly to measure the tidal influence in the lower basin.

Table 3-4: River flow, water level and reservoir water level gauges in K20A

Station	Record (Years)	Description	RATING
K2R002 - RES	27	Wolowedans Dam	139 m^3/s
K2H006 - DC	25	D/S River component	68 m^3/s
K2H002 - RIV	56	Gt – Brak @ Wolwedans	55.5 m^3/s
K2T004 - EST	29	Gt – Brak @ Vishoek	

Figure 3-3: Flow gauging stations and water level meters in K20A

3.4 Past hydrological analyses

There were numerous studies done by authorities and student studies alike, for the catchment area K20A. Some significant studies are highlighted in this section, and the recommendations of these studies will be discussed and considered in the flood estimation done for the flood routing calculations in this study.

Statistical information regarding the effects of the Wolwedans Dam on the potential floods was obtained from the initial study on the Great Brak Estuary (Council for Scientific and Industrial Research 1990). See Table 3-5 for the summary of the effect of the Wolwedans Dam on potential flood peaks.

Table 3-5: The effect of the Wolwedans Dam on the floods to the Great Brak estuary (Source: CSIR 1990)

Flood size	Before dam	After dam
Estimated maximum flood	2 100 m^3/s	1 918 m^3/s
Regional maximum flood	1 600 m^3/s	1 444 m^3/s
1:200 flood	1 100 m^3/s	961 m^3/s
1:100 flood	960 m^3/s	833 m^3/s
1:50 flood	800 m^3/s	686 m^3/s
1:20 flood	520 m^3/s	429 m^3/s
1:10 flood	320 m^3/s	254 m^3/s

A flood analyses undertaken by the Department of Water Affairs in 2011, focused on the flood occurrence at the Wolwedans Dam. A statistical approach was used to analyse the 50 largest events and to determine the size of floods with a certain exceedance probability of occurrence, see Table 3-6. The difference between Table 3-5 and Table 3-6 should be noted. Table 3-5 focusses on the floods into the Great Brak Estuary, and Table 3-6 on the floods entering the Wolwedans Dam alone.

Table 3-6: Exceedance probability of floods into the Wolwedans Dam (Source: (Council for Scientific and Industrial Research 2011)

EXCEEDANCE PROBABILITY (%)							
50	20	10	5	2	1	0.5	
FLOOD PEAKS (m^3/s)							RMF
55	225	350	435	555	650	745	1460

In 2012 a flood frequency analysis was done by the Directorate Hydrological Services for the Wolwedans Dam. The study only investigated the catchment area upstream of the Wolwedans Dam. Statistical methods were used, with input from a patched annual maximum inflow and outflow series, to assess the probable flood peaks for various return periods. The statistical flood peaks were recommended to be used as the flood peaks for the catchment (See Table 3-7). In the study, further investigation was done into the method of estimating flood hydrographs accurately for the catchment area. It was found that the Synthetic Unit Hydrograph method (SUH) significantly underestimates the extreme flood peaks in the area. The discrepancies in the flood peaks were attributed to the veld type zone 2, which the SUH method incorporates. The 2-t_c event Direct Run-off Hydrograph (DRH) method flood was found to be closest to actual past events and was recommended for use in flood routing calculations (Roux, Rademeyer 2012). A MAP of 730 mm was used to calculate extreme flood hydrographs for the Wolwedans Dam catchment area.

Table 3-7: Flood peaks recommended for the Wolwedans Dam catchment (Source: Adapted from Roux, Rademeyer 2012)

EXCEEDANCE PROBABILITY (%)							
50	20	10	5	2	1	0.5	
FLOOD PEAKS (m^3/s)							RMF
55	195	320	420	560	670	785	1460

3.5 Flood events

Historical flood events and estimated flood peaks will be discussed in this chapter. See Table 3-8 for the highest water levels recorded in the estuary since 1988 (Council for Scientific and Industrial Research 2011).

Table 3-8: Summary of the highest water levels recorded in the Great Brak estuary since 1988 (Source: CSIR 2011)

No.	Estuary Water Level @ K2T004	Flood Peak	Description	Date
1	2.913 m MSL	Inflow: 557 m^3/s Outflow: 336 m^3/s	Heavy rain; 79.2 mm in 24 hours with the Wolwedans Dam being at 88 % capacity before the rain event. Closed mouth conditions.	08/06/2011
2	2.429 m MSL	-	Waves overtopping the sand berm	01/09/2008
3	2.245 m MSL	Outflow: 245 m^3/s	Heavy rain and closed mouth conditions.	01/08/2006
4	2.24 m MSL	-	Waves overtopping the sand berm	25/05/2002
5	2.24 m MSL	Outflow: 145 m^3/s	Heavy rain	25/03/2003
6	2.194 m MSL	Inflow: 626 m^3/s Outflow: 404 m^3/s	Heavy rain, coinciding with open mouth conditions	22/11/2007

The highest water level recorded in the estuary was the event of heavy rains over a period of two days in June 2011. These rains coincided with a closed mouth condition and it was needed to perform an emergency breach. The berm height reached +3.8 to +4.5 m MSL in the prolonged closure of July 2009 to February 2011, and were also observed to be much wider than normal (Council for Scientific and Industrial Research 2011). This caused a dangerous situation, as the flushing efficiency was adversely affected by the large berm and caused damming of water in the lower estuary basin even when breaching occurred as per the EMP, i.e. at +2 m MSL.

The second and fourth highest recorded water level happened due to extreme wave conditions overtopping the berm across the river mouth. The waves overtopped a closed estuary mouth berm and increased the water level in the lower estuary basin to over +2 m MSL. It is clear that river floods and wave overtopping of the mouth berm can cause extreme water levels in the estuary.

The largest flood recorded to spill over the Wolwedans dam, since 1992, happened in November 2007. From Table 3-8, the flood recorded in the catchment (no. 6), have experienced some attenuation by the Wolwedans Dam. The flood into the dam correlates with a 100 – year storm in the catchment (See Table 3-6), but was reduced to 20 – year storm by the attenuation effect of the dam. The dam was at 65%

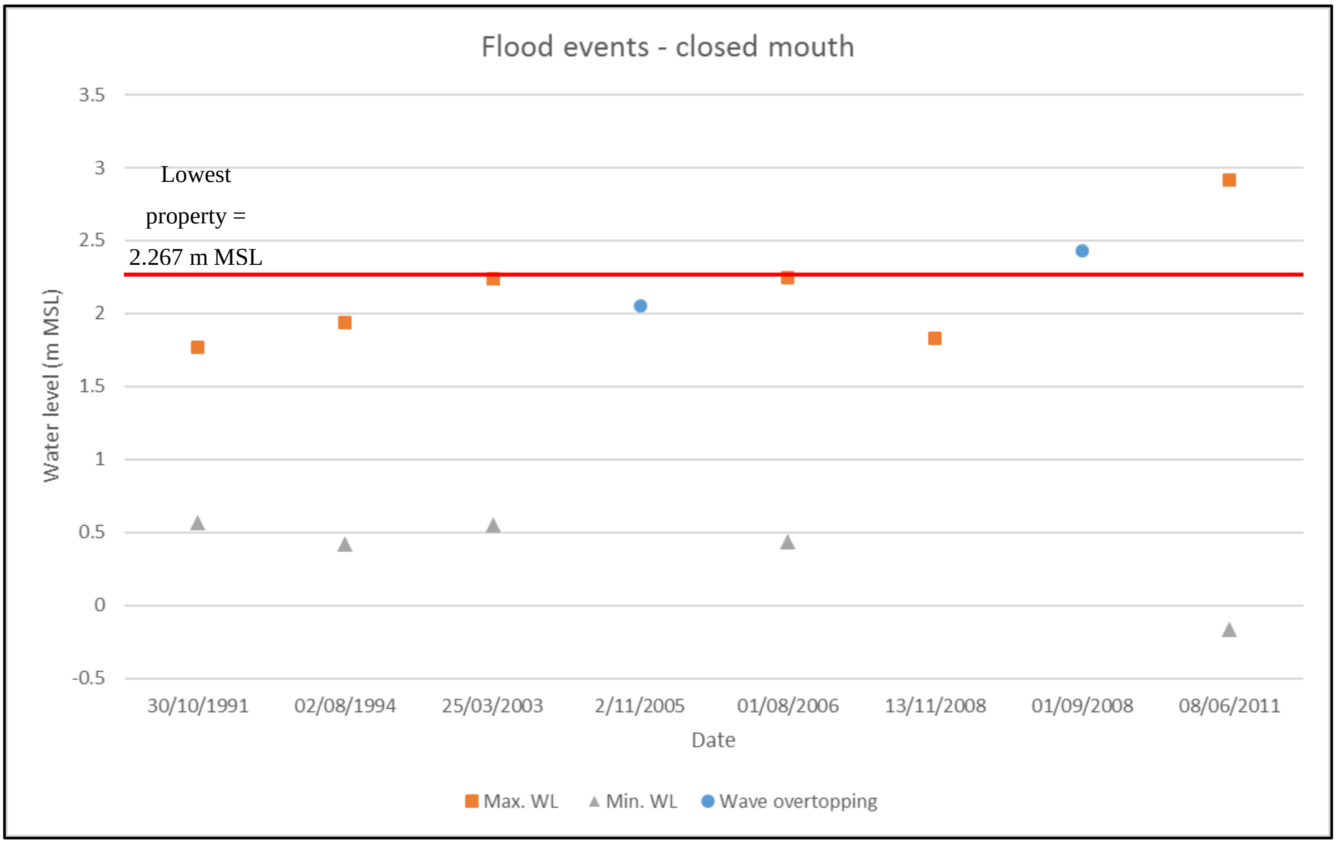


Figure 3-4: Maximum and minimum water level reached during and after flood events coinciding with closed mouth conditions

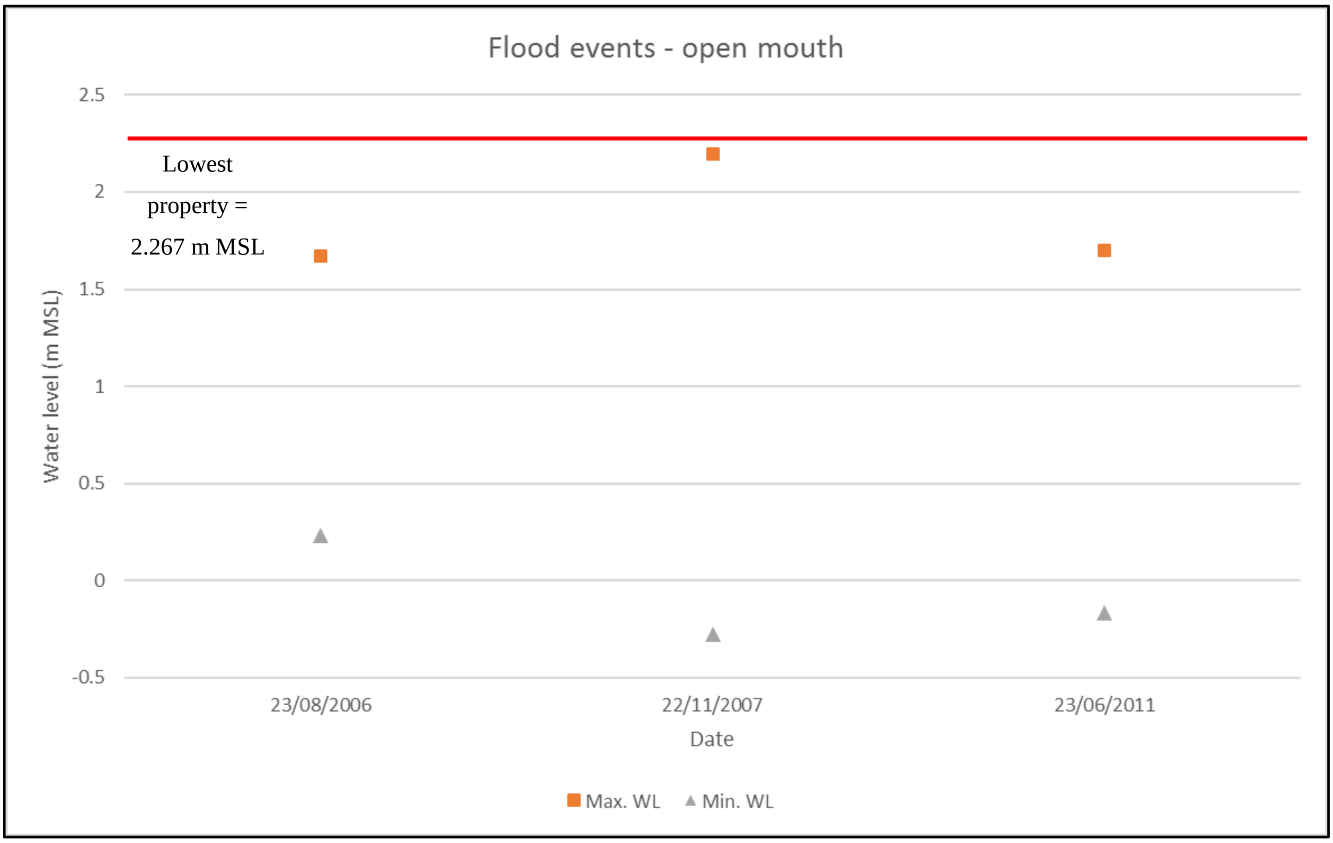


Figure 3-5: Maximum and minimum water levels reached during and after flood events coinciding with open mouth conditions

capacity before the storm. A flush release of 150 000 m³ to open the mouth was recorded merely two days before the flood and likely saved the low-lying properties in and around the estuary from flooding.

See Figure 3-4 (closed mouth) and Figure 3-5 (open mouth) for the maximum water level reached during the flood events and the minimum water level after the flood passed through the mouth. The minimum water level after the flood gives an indication to the minimum bed level of the inlet channel after the flood passed through the estuary mouth. From the figures, it is clear that more floods have coincided with closed mouth conditions than with open mouth conditions and that more severe water levels are reached when the mouth is closed. The lowest property in the lower estuary basin is at +2.3 m MSL (Council for Scientific and Industrial Research 1990) which is shown on Figure 3-4 and Figure 3-5 as a Red line.

3.6 Mouth states and manipulations

The mouth states of the Great Brak estuary as well as mouth manipulations since recording started in 1988 will be discussed in this section. The summaries were obtained from the Great Brak Estuary Monitoring Programme's latest and last instalment (Council for Scientific and Industrial Research 2011).

Table 3-9: Open mouth conditions since 1988 (Source: Council for Scientific and Industrial Research 2011 and Piet Huizinga 2017)

Year (days)	Calender year number of days open	As % of the year	Dam Overflow	Hydr. Year days Sept – April	Days open Sep (Previous year)- Apr	% Days open Sep-Apr	Total released for Hydr. Year (M^3)
1988 (240)	142 (pre-dam)	59		242	145	60	
1989 (365)	133 (pre-dam)	36		242	208	86	
1990 (365)	167 (filling started 7 May 1990)	46		242	34	14	
1991 (365)	57	16		243	51	21	
1992 (365)	124 (Overflowing on 16 October 1992)	34		242	209	86	
1993 (365)	248	68	Yes	242	212	88	1 750 280
1994 (365)	261	72	Yes	242	242	100	690 000
1995 (365)	276	75	Yes	243	204	84	N.A.
1996 (366)	182	50	Yes	242	210	87	N.A.
1997 (365)	337	92	Yes	242	220	91	732 800
1998 (365)	168	46	Yes	242	68	28	2 011 600
1999 (365)	174	48	Yes	242	176	73	2 092 640
2000 (365)	166	45	Yes	242	171	71	1 675 440
2001 (365)	233	64	Yes	242	229	95	2 850 000
2002 (365)	228	62	Yes	242	130	54	4 954 000
2003 (365)	258	71	Yes	243	147	60	3 934 000
2004 (366)	128	35	No	242	219	91	3 665 000
2005 (365)	231	63	Yes	242	117	48	1 461 600
2006 (365)	175	48	Yes	242	242	100	2 961 000
2007 (365)	225	62	Yes	242	217	90	1 945 000
2008 (366)	267	73	Yes	242	136	56	2 091 500
2009 (365)	37	10	No	242	0	0	0
2010 (365)	0	0	No	242	16	7	1 270 000
2011 (365)	224	61	Yes	243	243	100	619 500
2012 (366)	311	85		242	242	100	
2013 (365)	249	68		242	208	86	
2014 (365)	277	76		242	155	64	
2015 (365)	266	73		242	213	88	
2016 (91)	91	100					

From Table 3-9 can be seen, the Great Brak Estuary experienced open mouth conditions 56.5 % of the time between 1988 and 2016. All breaching's of the estuary mouth is summarised in Figure 3-6. The summary was compiled from Table B-1 in Appendix B, which was obtained from the Council for Scientific and Industrial Research (2011) for the period 1990 – 2011. The breaching events between 2011 – 2017 was updated from information obtained from DWA (2017).

Figure 3-6 is a summary of all types of mouth manipulations described in Section 3.2.2. Three types of breaching's were recorded from 1990 and onwards. Planned breaching's and emergency breaches were mostly assisted with a release from the Wolwedans Dam and were initially conducted at a +1.62 m MSL water level which increased to +2 m MSL after it was found that breaching at higher levels offers a larger head to increase the flushing efficiency. The outflow rate accompanying the breaches also increase from <10 m^3/s to 66 m^3/s (Council for Scientific and Industrial Research 2011). Flushing the mouth open a day or two after it closed was recorded as the third type of breaching and occurred at much lower water levels (approx. at +1.5 m MSL). These flushing's also happened more frequently than the planned and emergency breaching's.

Figure 3-6 shows three different recorded water levels relevant to a breaching event. The water level before release is the water level prior to a release from the dam, the water level after release is the maximum water level reached prior to breaching and the lowest water level after breaching is the lowest

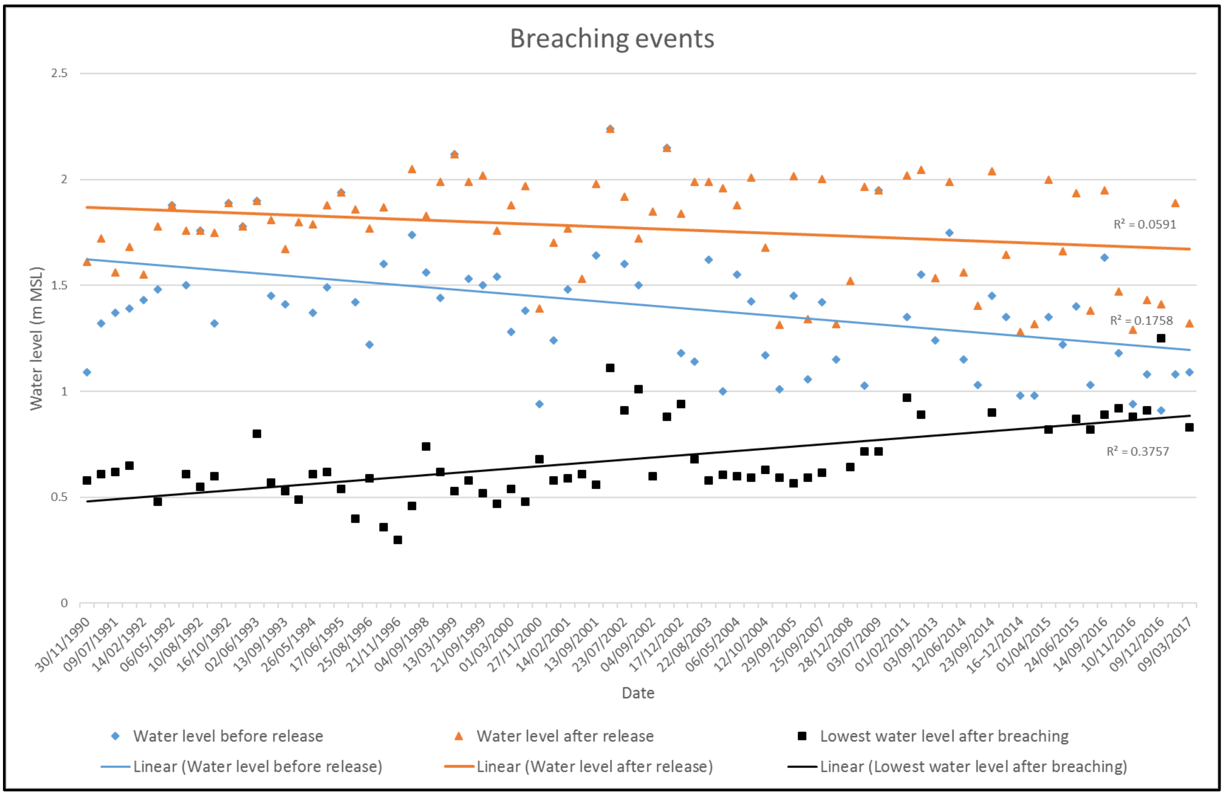


Figure 3-6: Summary of observed water levels in the lower estuary basin before, during and after all breaching events

draw down after a breaching event, indicating the bed level of the inlet channel. Linear trend lines were used to assess the trends of these water levels through the years. Although planned breachings occur at +2 m MSL, the frequency of the recorded flush openings is higher (there are two neap tide cycles per month) which affected the trend in the average water level at breaching negatively through the years. The minimum water level after breaching can give an indication of flushing efficiency. A positive trend is observed in the minimum water level reached after breaching, which can also be attributed to the increased frequency of flushes. Flushes occur at lower water levels which adversely affects the flushing efficiency.

Ensuring open mouth conditions for as long as possible seems to be the objective of the Authorities, which is easiest achieved by a flush. Flushing open the mouth also requires less water than a planned breach, however a compromise seems to be the amount of sediment flushed out of the estuary which may negatively affect the length of open mouth conditions and lead to another required flush release.

3.7 Tidal levels

The tide charts for Mossel Bay provides the expected tidal levels and can be seen in Table 3-10.

Table 3-10: The tidal levels at Mossel Bay (SANHO 2017)

Parameter	Level (m CD)	Level (m MSL)
Lowest astronomical tide (LAT)	0	-0.933
Mean low water springs (MLWS)	0.26	-0.673
Mean low water neaps (MLWN)	0.88	-0.053
Mean level (ML)	1.17	0.237
Mean high water neaps (MHWN)	1.46	0.527
Mean high water springs (MHWS)	2.10	1.167
Highest astronomical tide (HAT)	2.44	1.507

The levels are given relative to Chart Datum or the Lowest Astronomical Tide, and relative to Mean Sea Level or Land Levelling Datum (South African Navy Hydrographic Office 2017).

These values show that the tidal range (between MLWS and MHWS) for Mossel Bay is 1.84 m and correlates with microtidal (0 - 2m) conditions (See Section 2.3.1). These values do not account for any storm surge or other geological and meteorological conditions (South African Navy Hydrographic Office 2017).

3.8 Wave climate

The wave climate on the south – western and southern coasts of South Africa have been described as having the most severe wave conditions by Rossouw and Theron, 2009. The magnitude of wave action decreases along the western and eastern coasts moving northward (Rossouw, Theron 2009).

Three different types of synoptic patterns significantly influence the weather and wave climate around the coast. The waves observed on the western and south-western coasts are generally generated from passing cold frontal systems originating in the south – Atlantic. The waves observed on the southern to eastern coast are sometimes generated by cut-off low systems whereas the high waves observed on the eastern coast are on rare occasions generated by the occasional tropical cyclone off the Mozambican coast (Rossouw, Theron 2009).

Directional wave data is available from a site roughly 85 km south of Cape St. Blaize, the southernmost headland of Mossel Bay. See Figure 3-7, for the seasonal wave roses compiled from the full dataset by Hugo (2013). The wave dataset covers a 10-year period from January 1997 to June 2008 and was obtained from the United States National Centre for Environmental Prediction (NCEP). This data does not reflect real measurements, as the wave conditions were calculated from the results of the numerical climate model, WAVEWATCH III (Hugo. 2013). From Figure 3-7 can be seen that the prevailing wave direction is from the south-western quarter (220˚ - 225˚).

The nearshore wave climate, in Mossel Bay, is continuously measured by a nondirectional WAVERIDER buoy, belonging to the CSIR. The data of this measuring instrument was used by Clarke (2016) to determine the extreme wave conditions in the bay and will be discussed further in subsequent chapters. This buoy is also used to update the WaveNet model by CSIR, which is monitored by the Authorities when assessing the key parameters discussed in Section 3.2.4.

Rossouw (1989) determined the extreme significant wave parameters for deepsea conditions from Waverider records for most of the southern African coast, from Luderitz on the West coast to Richards bay on the East coast. The Sedco K recording station, located roughly 120 km offshore of Mossel Bay in 100 m of water is chosen as the location that is representative of the offshore wave climate at Great Brak (Rossouw. 1989). A set of wave heights for extreme return period events were obtained from the CSIR (von St. Ange. 2017 pers. comm.) for offshore locations around the South African coast. The FA site at the Agulhas bank was deemed to be representative of the offshore conditions at Mossel Bay. These extreme wave parameters will subsequently be used as the deepsea design wave parameters.

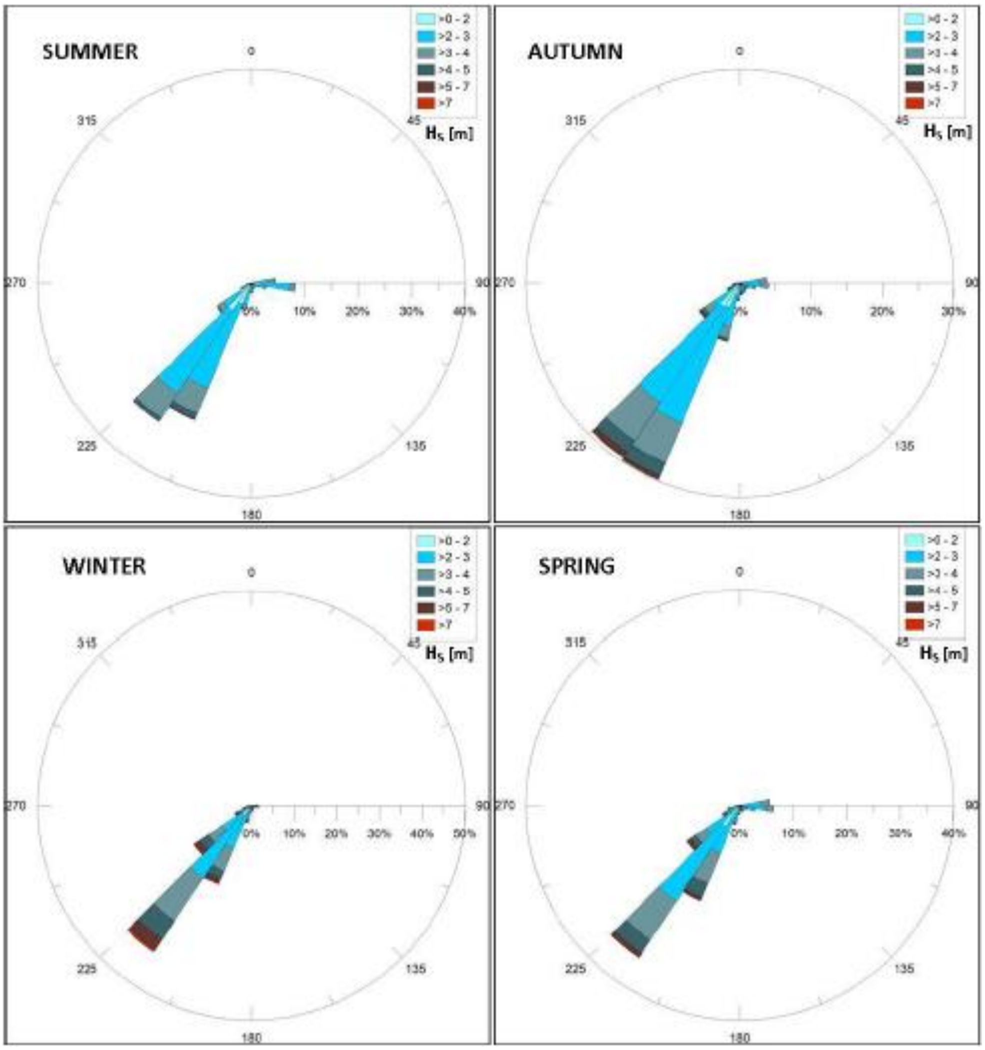


Figure 3-7: Seasonal distribution offshore wave roses (Source: Hugo 2013)

3.9 Surveys

From the start of the estuary monitoring programme in 1990, various cross-sectional surveys of the estuary reach and contour plotting of the area downstream of the railway bridge (lower estuary reach) have been captured. The most recent survey was done in 2005. From Figure 3-8 can be seen that the estuary is fairly shallow and that the channel around eastern side of the Island is deeper than the channel around the south-western tip of the Island. This confirms reports that the main flow of water is around the eastern side of the Island (See Figure 3-8). The detailed survey only measured the outside perimeter of the island. The south-eastern part is slightly higher (7 m MSL at one point) with a steep gradient, where the perimeter elevations from the western part around the northern side to the eastern side of the island is considerably lower (2.2 – 3.5 m MSL range). The majority of the Island perimeter is under +5 m MSL.

Three other detailed surveys of the lower estuary basin were done in 1988, 1993 and 2001. See Figure A-1 to Figure A-3 in Appendix A for the three surveys. The eastern channel around the Island is deeper than the southern channel, and seem to be the dominant connection to the ocean. A difference plot (Figure A-4) was developed between the 1988 and 2005 surveys. It shows the dynamic nature of

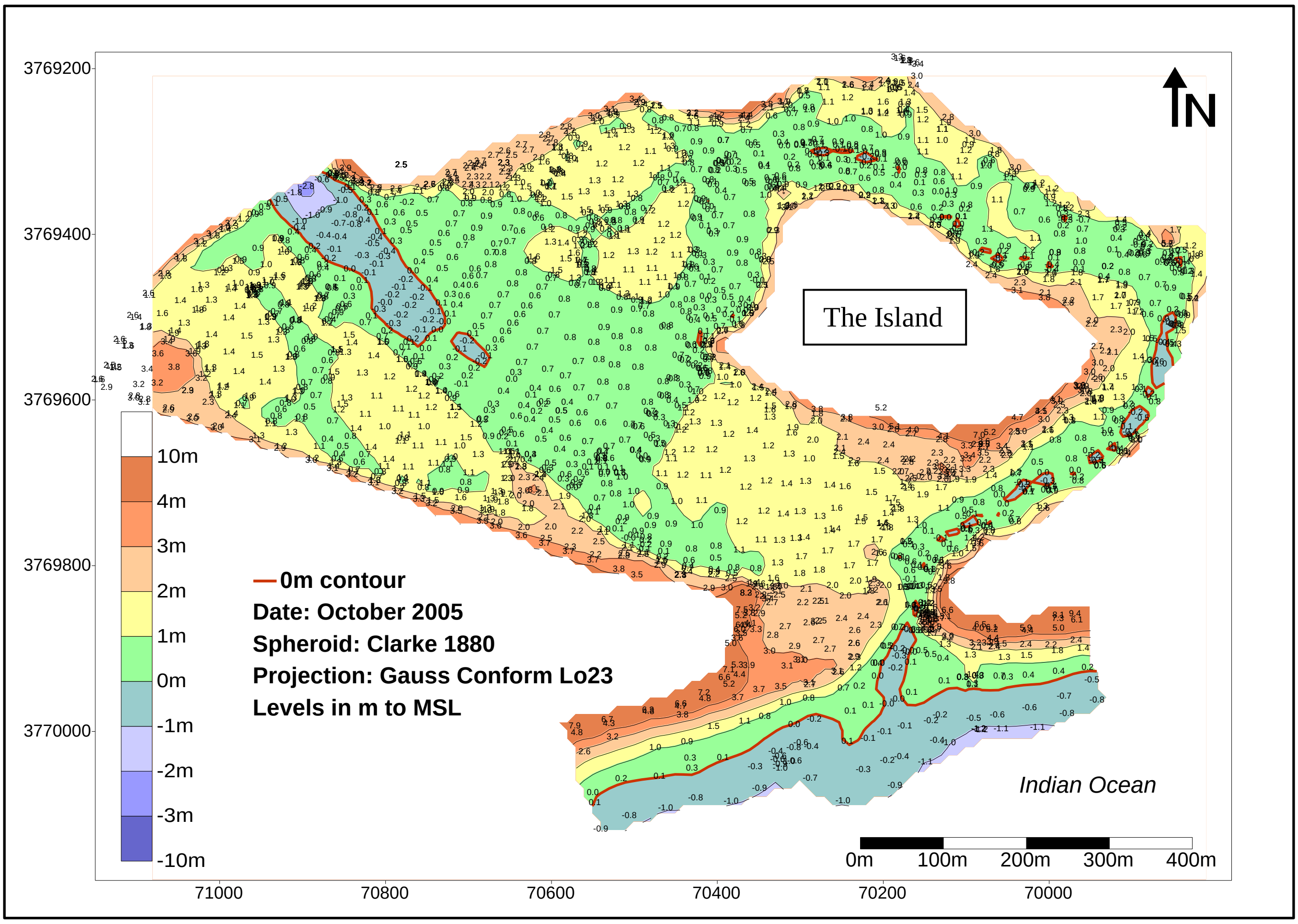


Figure 3-8: Bathymetric survey of the lower estuary basin done in 2005 (Source: Huizinga 2017)

Figure 3-9: Channel around the Island (Source: Google Earth 2014)

estuarine environments. No major or systematic changes have occurred, sediment has built up in some places and has been eroded in other places, as expected near an estuary mouth.

3.10 Modelling of the Great Brak Estuary

The Great Brak Estuary has been subjected to various studies since the decision was made to build a dam in the catchment. Two recent studies, done by Pieterse (2014) and by du Pisani (2015) on possible flood lines around the estuary are relevant to this study. Du Pisani (2015) assessed the adequacy of a 5-m setback line at the estuary, considering future climate change predictions, where Pieterse (2014) assessed the influence of storm surge and climate change on the 100-year flood lines of the estuary.

Both studies modelled the estuary using river modelling and analysis software. Pieterse (2014) used the HEC-RAS utility package and du Pisani (2015) utilised Mike11 by DHI. These studies are useful in the context of this study, as water levels in the lower estuary basin, where the Island is located, were calculated under extreme conditions. Both overland flooding and extreme sea conditions were assessed. The variability of the mouth state and the numerous bridges across the estuary were also addressed in both studies. The two studies, however, does not report on the potential flow velocities in the lower estuary basin that accompany flood events as the focus fell on flood line determination.

A short summary of the assumptions made, input parameters and the results of both studies will be discussed in this section.

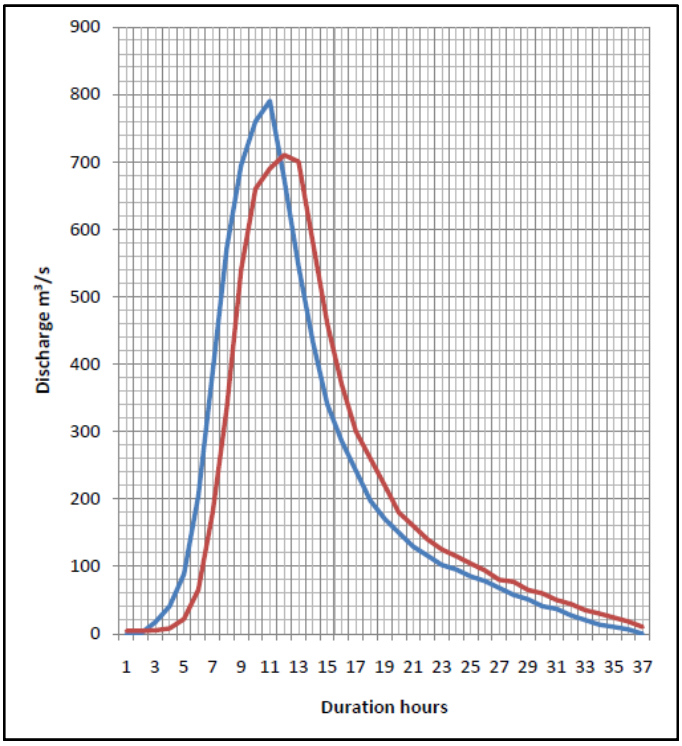
3.10.1 Pieterse (2014)

Pieterse (2014) conducted a study with the title: *The Influence of storm surge and climate change on the 100-year flood lines in the lower Great Brak Estuary*. The HEC-RAS 1-Dimensional modelling software was used in this study to calculate potential water levels in the estuary under extreme conditions. Twenty-six cross-sections of the lower and upper reaches of the estuary were developed using a topographic contour map, photogrammetric techniques and using a bathymetry contour map of the estuary. The defined river channel and the adjacent floodplains were included in the cross-sections.

The 100-year design flood for the catchment was calculated for the quaternary catchment area by means of further dividing the catchment area into two areas, above and below the Wolwedans Dam, understanding that the reservoir will impose an attenuation factor on the passing flood. The dam was assumed to be at Full Supply Capacity (FSC), as the worst-case scenario was to be investigated. See Figure 3-10 for the inflow and outflow hydrographs for the Wolwedans Dam (Left), and then the inflow hydrograph for the Great Brak Estuary (Right). A peak discharge of 764 m^3/s was used for the purposes of the model.

The water levels, on the ocean side, considered sea level rise due to climate change and positive storm surge. A water level of +3.44 m MSL was calculated for the situation where a storm surge of 1.14 m, a sea level rise of 0.8 m both coincide with a Highest Astronomical Tide of 1.507 m MSL (Pieterse 2014).

The model was then calibrated with the storm event that occurred over the period 22 – 26 November 2007 and the necessary errors were corrected for. The model was set up to consider the artificial breaching activity at the estuary, as set out in the Management Plan. Thus, for the case where the mouth is closed, a +2.0 m MSL control level at the lower estuary was used, where the breaching channel increases in width, as scouring increases with the flow in the channel (Pieterse 2014). A few scenarios were modelled, namely the event of a closed mouth coinciding with a 100-year flood as well as the



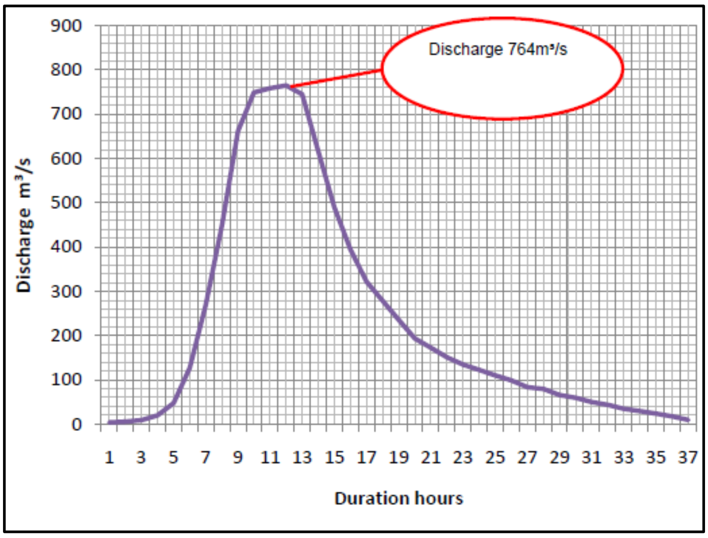


Figure 3-10: (Left) Inflow and Outflow Hydrograph for the Wolwedans dam (Right) Inflow hydrograph for Great Brak Estuary (Source: Pieterse 2014)

event of an open mouth, 100-year flood conditions and with sea level rise and storm surge from the ocean. The ocean condition was modelled as a fixed downstream water level boundary condition, making it a rather rigid assumption; i.e. no tidal variation or change in surge height.

The results were interesting, the closed mouth condition showed a maximum water level of +3.3 m MSL at the lower estuary and +3.9 m MSL in the upper estuary basin. For open mouth conditions, maximum water levels of +3.6 m MSL and +4.1 m MSL for the lower and upper estuary basins respectively were achieved. See Figure 3-11 for the calculated flood lines. During both extreme scenarios, the Island will experience flooding, more so with the open mouth condition (RED).

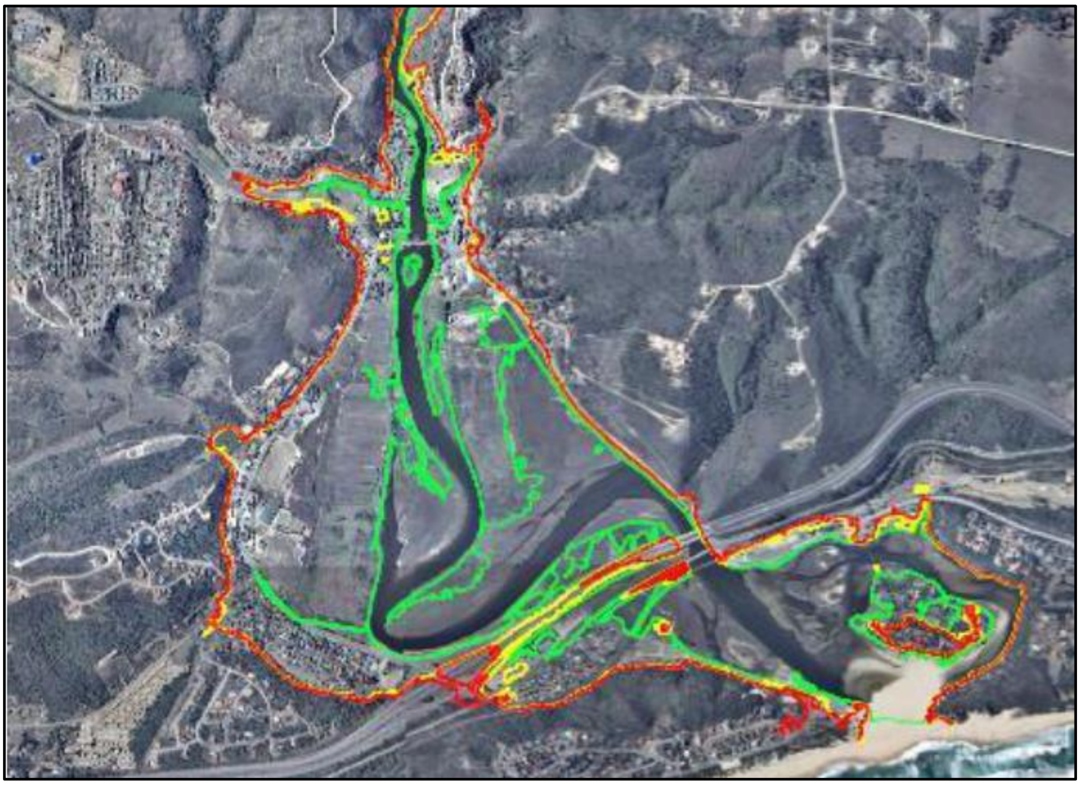


Figure 3-11: Simulated flood lines for three situations: (GREEN) Estuary control WL +2.0 m MSL, (YELLOW) 100-year flood event with breaching @ WL +2.0 m MSL, (RED) 100-year flood event with open mouth, SLR and storm surge (Pieterse 2014)

3.10.2 Du Pisani (2015)

The title of the du Pisani (2015) study is: *The effect of sea level rise on flood levels in the Great Brak Estuary: assessing the adequacy of a 5-m setback line*. This study uses the 1-Dimensional hydraulic modelling software Mike11 to model the Great Brak river from the Wolwedans Dam to the estuary mouth. The river was modelled by 81 cross-sections, with maximum spacing of 180 m. The defined river channel was included, as well as meandering channels, included as “links” to the defined channel, e.g. the meandering channel around the Island.

The effect of the vegetation on the floodplain banks were included as different bed roughness coefficients. The correct roughness coefficients were obtained from a trial and error calibration process, using a short period of measured data where there were relatively low and high flows. The bridges and its abutments were also included as cross-sections. Careful assumptions were also made for the influence of sediment deposits in three places, above and below the Searle's, and the N2 & railway bridges and in the lagoon, westward of the Island. The Island was modelled as an area with a fixed bed.

The upstream inflow boundary condition, at the Wolwedans Dam, was a calculated inflow hydrograph. Like Pieterse, du Pisani (2015) calculated the 100-year flood hydrograph for the catchment above and below the Wolwedans Dam, assuming the dam is at FSC. A flood routing technique was incorporated to include the attenuation effect of the dam. A peak flow of 800 m^3/s was calculated. See Figure 3-12 for the 100-year flood hydrographs calculated in this study.

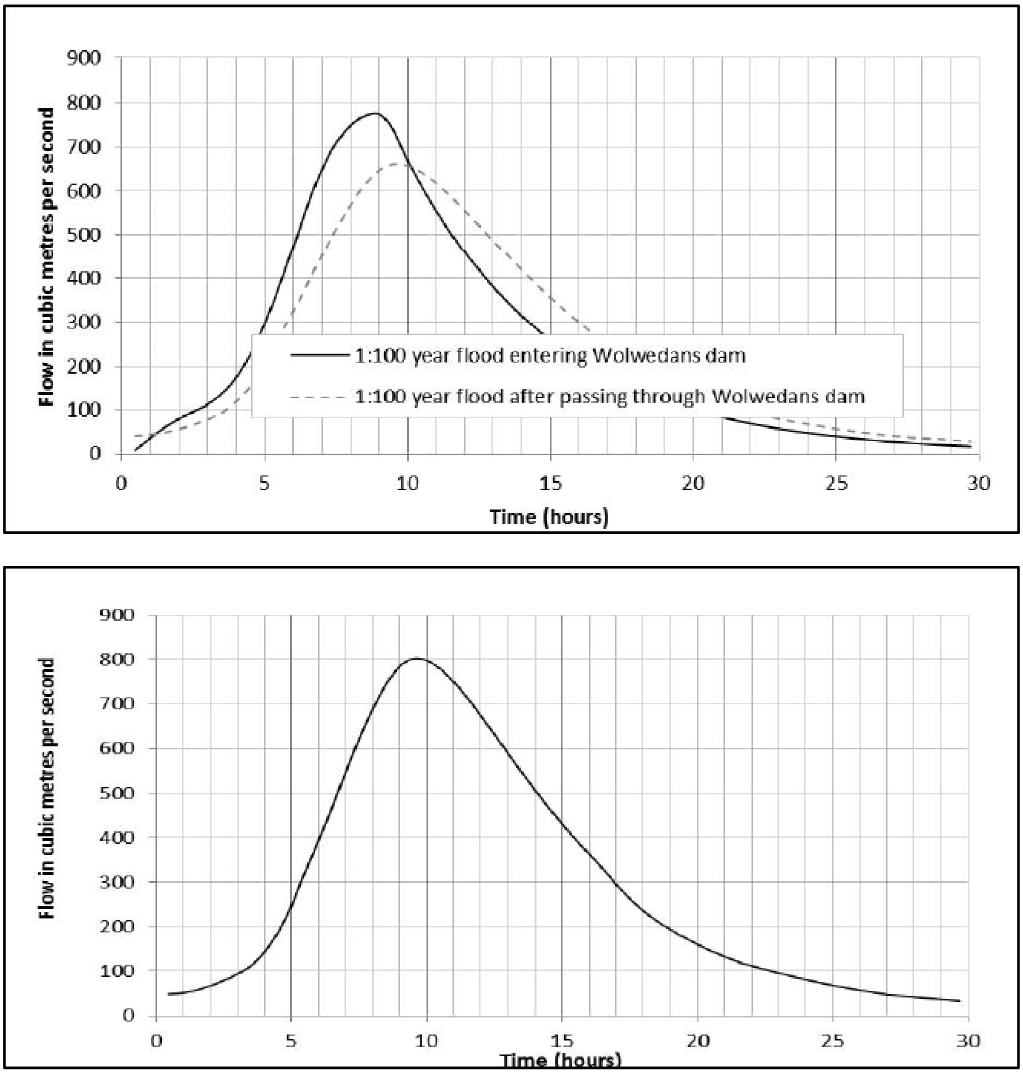


Figure 3-12: (ABOVE) Inflow and outflow hydrographs for the Wolwedans Dam (BELOW) Inflow hydrograph into the Great Brak Estuary (du Pisani 2015)

The mouth of the estuary was modelled as a fixed and raised bed cross-section for varying situations, to model open and closed mouth conditions. The ocean was modelled using a time series boundary condition for the open mouth case for berm bed levels of 0 m MSL and -1.0 m MSL, to include tidal effects of the ocean. The berm was treated as a fixed barrier in the case of a closed mouth condition. The maximum or design ocean water level was calculated as +2.67 m MSL. The water level incorporates storm surge, storm surge rise, sea level rise; all of which are superimposing on a MHWS event. Wave energy inside the estuary was assumed to be negligible, as the sandy berm was deemed to dissipate all wave energy. Wind set-up inside the estuary was also neglected.

This study also included a simulation, to see the estuary water level response to the barrier height at the estuary mouth, where the mouth did not get breached at the control water level of +2.0 m MSL, to simulate closed mouth conditions. The berm heights from +1 m to +4 m MSL were simulated, as a downstream barrier. This simulation is relevant, as the berm height can reach levels close to +4.0 m MSL on periods of extended low river flows, and as the berm height is subject to sea levels and run-up levels which is predicted to increase due to climate change. See Figure 3-13 for the results of these simulations (Du Pisani. 2015).

For the open mouth simulations, a few scenarios were also investigated. Various downstream water levels were used as input to simulate different ocean storm levels. These simulation results can be seen in Figure 3-14 to Figure 3-16. The water level between chainages 7300 and 8000 is relevant for this study, as this is where the Island is situated. For the scenarios, where the 100-year flood coincides with

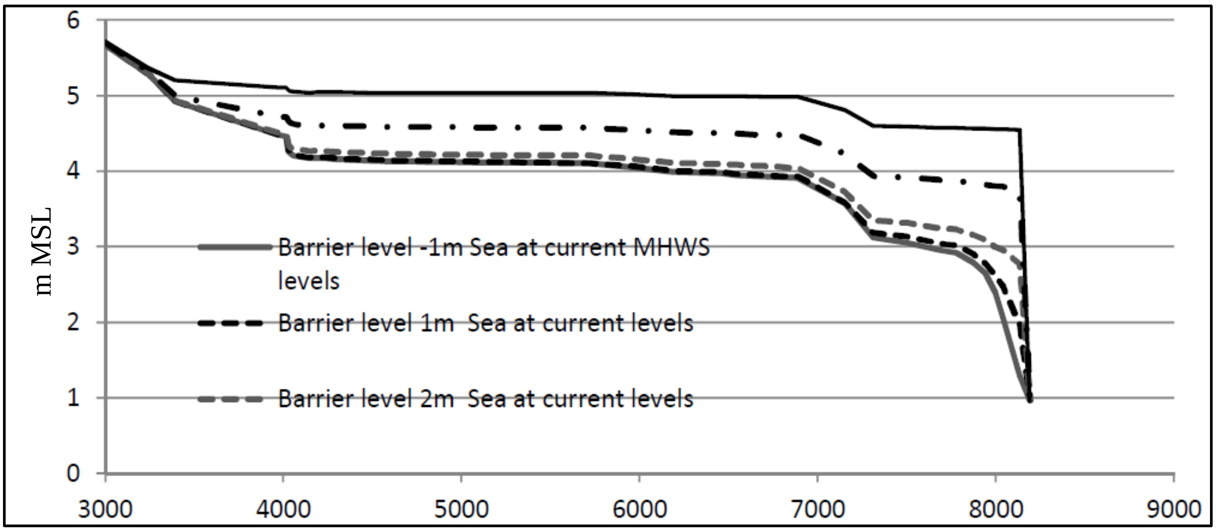


Figure 3-13: Estuary water level vs chainage, simulating a response to a 100-year flood event and a barrier of varying height downstream (Source: du Pisani 2015)

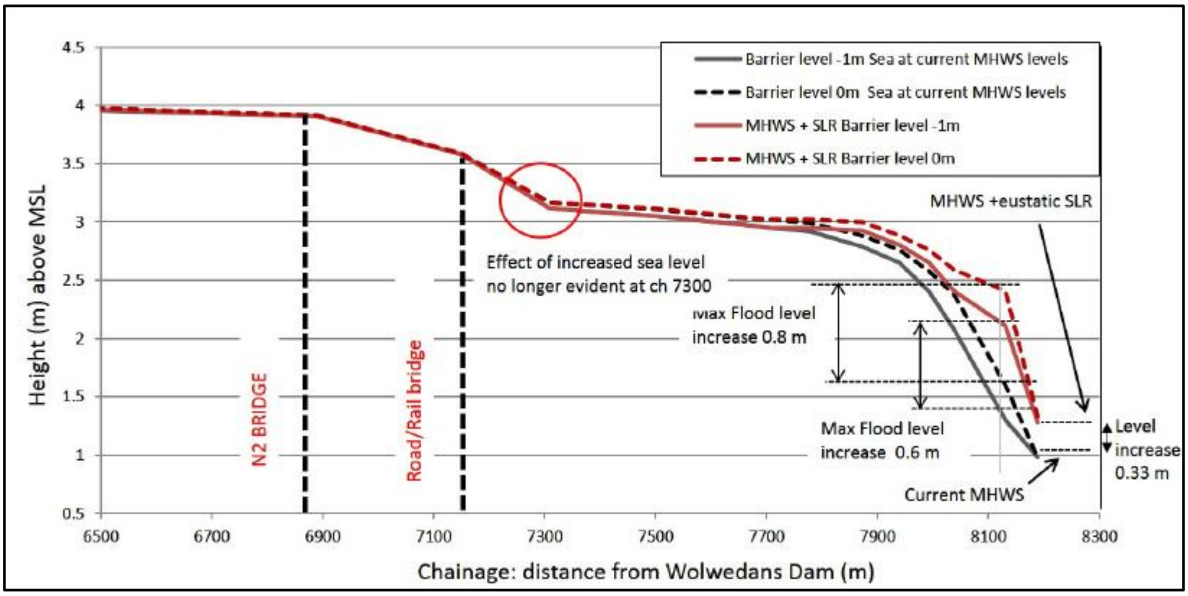


Figure 3-14: Estuary water level vs chainage, simulating a response to a 100-year flood event coinciding with a MHWS event plus SLR of 0.33 m (Source: du Pisani 2015)

current MHWS and MHWS plus a sea level rise of 0.33 m, for barrier heights of 0 m MSL and -1 m MSL, see Figure 3-14. A maximum level of +3.1 m MSL for both cases were reached in the lower estuary basin.

For the scenarios where a 100-year flood coincides with a MHWS event, with storm surge and a MHWS event plus storm surge and SLR for barrier heights of 0 m MSL and -1 m MSL, see Figure 3-15. Again, water levels of over +3.0 m MSL were achieved in the lower estuary basin for these events. A certain amount of turbulence, for the MHWS + SLR + Storm surge event, at the estuary mouth was also noted, and treated as unreliable water level results.

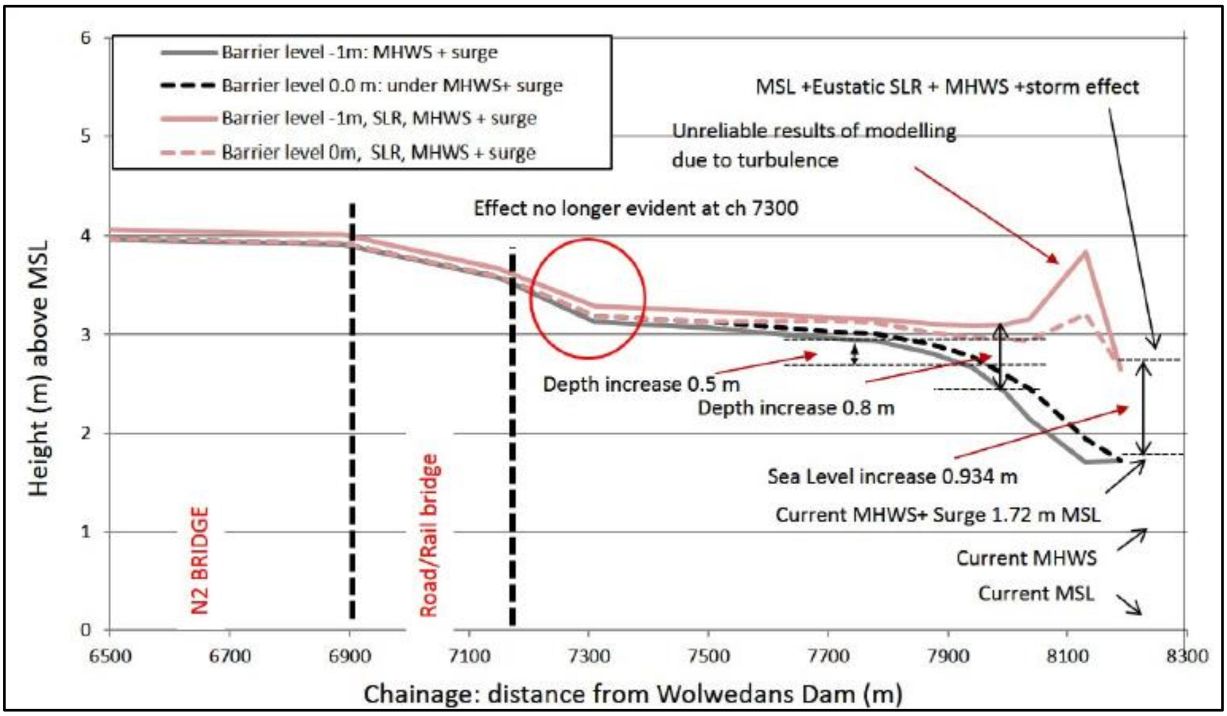


Figure 3-15: Estuary water level vs chainage, simulating a response to a 100-year flood event coinciding with a MHWS event plus SLR of 0.934 m and a storm surge component (Source: du Pisani 2015)

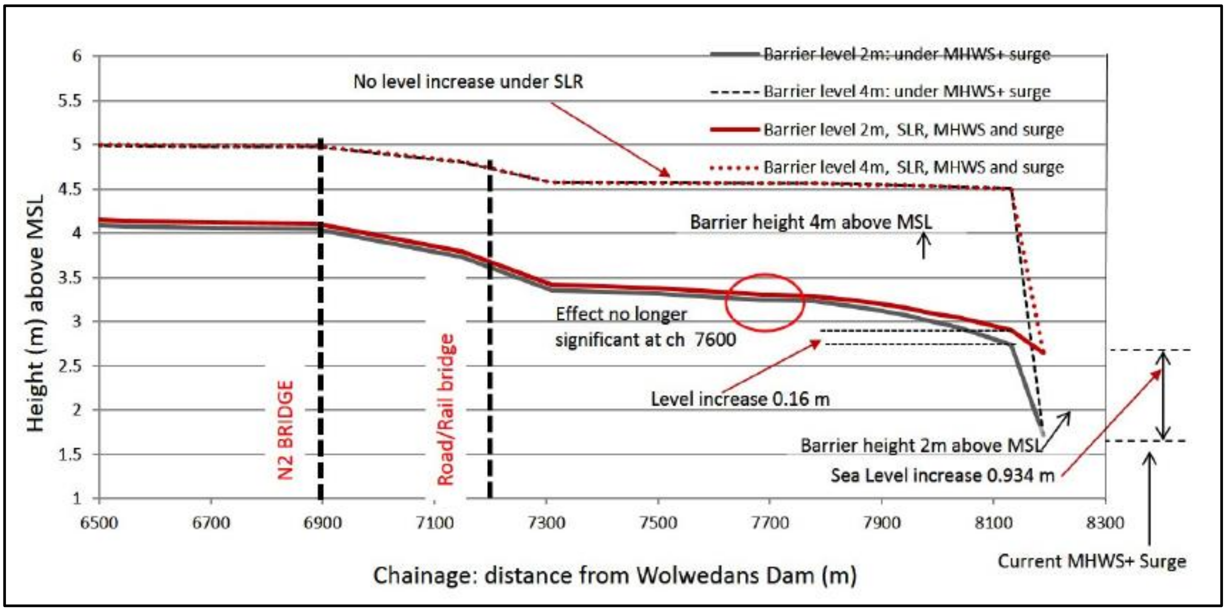


Figure 3-16: Estuary water level vs chainage, simulating a response to a 100-year flood event coinciding with a MHWS event plus SLR of 0.934 m and a storm surge component (Source: du Pisani 2015)

The MHWS coinciding with storm surge, with and without SLR, were simulated again, this time for the berm heights 2 m MSL and 4 m MSL, see Figure 3-16. In both cases, the 2 m MSL barrier level model resulted in water levels between +3.0 and +3.5 m MSL for the lower estuary basin. The 4 m MSL barrier level model predicted water levels between +4.5 m MSL and +4.6 m MSL.

3.10.3 Summary

The results of the two studies discussed in the preceding sections were summarised for ease of comparison. The division between the lower and upper estuary reaches were made, as the Island is situated in the lower estuary reaches. The divide is chosen as the N2 bridge across the estuary, which is just upstream of the Island.

See Table 3-11 and Table 3-12, for the summarised results of the two studies. It is important to note that under all simulated scenarios, the Island in the lower reaches will experience extensive flooding. The findings of the two studies echo the concern of flooding of property in the estuary. The influence of sea level rise and predicted storm surge rise, as highlighted by du Pisani (2015) will have an influence in the lower reaches of the estuary.

Neither modelling effort considered the dynamic nature of the inlet geometry in width nor depth. This is a limitation to the results, as the outflow might be underestimated and the extreme water levels overestimated. The rigid mouth geometry assumption can be viewed as a worst-case scenario and can be accepted as a first order estimate of obtainable extreme water levels in the absence of more detailed hydrodynamic modelling of the estuary reach and mouth conditions. The reliability of the results is uncertain as the two modelling efforts were performed by inexperienced individuals with software packages that are not adequate to take into account the dynamic nature of the mouth morphology and the river run-off and ocean interaction.

Table 3-11: Summary of the simulation results from Pieterse (2014)

Pieterse (2014) – HEC RAS	
Scenario	Calculated water levels
1. Closed mouth, 100 – year flood, SLR, Storm surge, superimposed on HAT, artificial breaching as per Management Plan	• Lower Estuary basin = **+3.3** m MSL • Upper Estuary basin = **+3.9** m MSL
2. Open mouth, 100 - year flood, SLR, Storm surge, superimposed on HAT	• Lower Estuary basin = **+3.6** m MSL • Upper Estuary basin = **+4.1** m MSL

Table 3-12: Summary of the simulation results from du Pisani (2015)

du Pisani (2015) – Mike11	
Scenario	Calculated water levels
1. 100-year flood, closed mouth (treated as barrier at various heights) – barrier heights (-1 m + MHWS, +1 m, +2 m, +3 m, +4 m (MSL))	Lower Estuary Basin: Barrier @ -1 m MSL+MHWS = +3.1 m MSL Barrier @ +1 m MSL = +3.2 m MSL Barrier @ +2 m MSL = +3.4 m MSL Barrier @ +3 m MSL = +3.9 m MSL Barrier @ +4 m MSL = +4.6 m MSL Upper Estuary Basin: Barrier @ -1 m MSL+MHWS = +4.1 m MSL Barrier @ +1 m MSL = +4.2 m MSL Barrier @ +2 m MSL = +4.3 m MSL Barrier @ +3 m MSL = +4.7 m MSL Barrier @ +4 m MSL = +5.2 m MSL
2. 100-year flood, Open mouth (Various barrier levels), coinciding with a MHWS event and a MHWS event plus SLR of 0.33 m.	Lower Estuary Basin: Barrier @ -1 m MSL+MHWS = +3.1 m MSL Barrier @ 0 m MSL+MHWS = +3.1 m MSL Barrier @ -1 m MSL+MHWS+SLR = +3.3 m MSL Barrier @ 0 m MSL+MHWS+SLR = +3.4 m MSL
3. 100-year flood, open mouth (Various barrier levels), coinciding with a MHWS event, storm surge and a MHWS event plus storm surge (SS) and SLR of 0.934 m.	Lower Estuary Basin: Barrier @ -1 m MSL+MHWS+SS = +3.1 m MSL Barrier @ 0 m MSL+MHWS+SS = +3.1 m MSL Barrier @ -1 m MSL+MHWS+SS+SLR = +3.3 m MSL Barrier @ 0 m MSL+MHWS+SS+SLR = +3.4 m MSL
4. 100-year flood, at +2 m and +4 m barrier levels for MHWS+SS and MHWS+ SS+SLR	Lower Estuary Basin: Barrier @ +2 m MSL+MHWS+SS = +3.35 m MSL Barrier @ +4 m MSL+MHWS+SS = +4.6 m MSL Barrier @ +2 m MSL+MHWS+SS+SLR = +3.45 m MSL Barrier @ +4 m MSL+MHWS+SS+SLR = +4.6 m MSL

3.11 Conclusion

The situation of the Great Brak estuary has been described in preceding sections. The lower reaches of the estuary have been developed extensively and many low-lying properties within and adjacent to the estuary are vulnerable to flooding. The development of the Wolwedans Dam on the Great Brak river has greatly modified natural run-off conditions and an Estuary Management Plan has been developed which by means of a Water Release Policy from the Wolwedans dam, meets the Ecological Water Requirement of the estuary. Some important conclusions can be drawn from the situation assessment as discussed in the following sections.

3.11.1 Mouth states and manipulations

The estuary mouth has been manipulated for over two centuries due to flooding of low-lying human development. Flooding of the causeway was the first form of infrastructure to necessitate estuary mouth manipulation. Since the Wolwedans Dam was completed, the estuary mouth has been opened by mechanical means (bulldozer) or flushing by a volume release from the upstream dam (or both). The EMP stipulated guidelines for mouth openings and water releases from the dam. The EMP recommends the opening of the estuary mouth should be centred around the Spring and Summer periods. The estuary should be allowed a minimum of $1 \times 10^6\ m^3$ of water per year to ensure the opening of the mouth area.

The desired initial water level for a planned breaching have raised from +1.62 m MSL to +2.0 m MSL as the flushing efficiency increased as the head of water before breaching increases. A $0.5 - 0.75 \times 10^6\ m^3$ volume release from the dam normally provides enough water to perform a planned breach. An increase in maximum observed outflow have been noticed for higher initial water levels before breaching. The estuary mouth tends to close during neap tides, when the tidal action cannot keep the mouth open. Low flow releases during neap tides or a larger flush of water a day or two after closure will keep the mouth open. The object of breaches is to open and keep the mouth open for as long as possible. The mouth has experienced open mouth conditions 56.7 % of the time since 1988, as result of the EMP recommended planned breaches.

The estuary mouth state and berm height have proved to be truly dynamic and an important hydraulic control. During extreme rainfall events in the catchment, an already open mouth can spare the lower reaches of the estuary from flooding, as the water flowing over the dam can flow directly into the ocean whereas if the estuary mouth had been closed, the water would dam up and cause flooding of low-lying properties. During extreme ocean storm events, a closed mouth can protect the properties on the Island from the tidal and wave intrusion; however, if the closed estuary berm is overtopped by large waves, it can also cause high water levels in the estuary.

3.11.2 Flood events and water levels

The largest flood experienced in the catchment was a 1 in 100-year storm in 2007. The dam was at 65% before the rain started and proceeded to fill up and spill. The flood peak flow over the dam wall was 404 m^3/s, reduced from a 100-year storm to a 20-year storm peak flow into the estuary basin. The flood coincided with an open mouth and did not cause any significant flooding of the low-lying properties (lowest property located at +2.27 m MSL). A water level of +2.19 m MSL was reached in the lower estuary basin and a maximum draw down of -0.28 m MSL was recorded. This is the lowest recorded water level in the estuary basin after a significant river flood or breaching event.

The highest water level ever recorded in the estuary happened in 2011, when 79.2 mm of rain fell in 24 hours and caused a peak flow into the dam of 557 m^3/s, correlating to a 50-year storm. The dam level was at 88% and caused a significant attenuation of the flood peak flow into the estuary, which reduced the peak flow into the estuary to a 10-year storm. The storm run-off coincided with a closed mouth and caused major flooding of low-lying properties in the estuary. A water level of +2.9 m MSL was reached during this flood, even though an emergency breach was performed as per the Estuary Management Plan.

The upstream hydraulic control, the Wolwedans Dam, has a significant impact on passing floods. The dam has not been designed for flood control (Huizinga 2017 pers. comm.), therefore the flood routing calculations have always been done for an extreme flood event coinciding with a 100% full dam. A minimum attenuation of 13 % is expected for the 100-year flood peak and a full dam. Nevertheless, the dam has proved to play an essential role in attenuation of the floods if it is not 100% full.

The downstream hydraulic control, the estuary mouth state, mouth (width and depth) and berm geometry (elevation and length), is just as important in preventing extreme water levels in the estuary. Following periods of severe drought, the berm has built up to be higher and wider than normal which lowered the flushing efficiency of an emergency breach, and caused excessive damming in the estuary (the flood of 2011). During ocean storms, the estuary can be protected from wave penetration by the berm during closed mouth conditions. It can also experience flooding caused by wave overtopping of the closed berm.

Overtopping of the mouth berm occurred on two occasions, in 2008 and in 2002, while the mouth was closed and was the cause of the second and fourth highest water level ever recorded in the lower estuary basin. Water levels of +2.43 m MSL and +2.24 m MSL was achieved during these overtopping events. The closed mouth does however provide adequate protection for the Island from direct wave attack during these large wave events.

Due to climate change effects, i.e. SLR and an increase in storminess, predicted for the coming century, coastal flooding due to storm surge is deemed to become more frequent and the possibility of large waves causing damage to the properties on the Island is deemed to increase as the mean sea level rises. A structure designed for flood alleviation will need to consider the fluvial and marine driven flooding and be adequately protected from expected wave attack.

3.11.3 Estuary Management Plan

The EMP was developed for the estuary with the main objective of keeping the estuary health and ecological status at pre-dam conditions. The development of the EMP was done in conjunction with multiple site specific hydrodynamic, hydrologic, ecological and socio-economic studies with adequate public participation.

The EMP specified guidelines for activities like breaching, estuary ecological water requirement and a water release policy. The EMP will be taken as a best-practice guideline in the feasibility study of potential flood defence measures at the Island. Any future flood defence measures planned for the low-lying properties should comply to the three main objectives of the EMP:

- Maintain the ecosystem as close as possible to natural state
- The aesthetic quality of the estuary and the tidal influence in the lower reaches of the estuary had to be maintained
- Maintain the potential recreational value of the area, especially during peak season

The emergency protocol followed by the Authorities is described in Section 3.2.4 and is based on the identification of emergency conditions. The emergency conditions are identified by monitoring key elements surrounding the estuary. Weather reports from the SAWS are monitored for rainfall in the catchment and the ocean conditions are monitored from WaveNet by CSIR. If severe weather conditions are expected, the estuary mouth state and berm height is surveyed, the dam level and water level in the estuary is assessed. A decision to release water or to mobilise breaching equipment is subsequently made as a proactive measure to prevent extreme water levels during the flood event.

4. Potential flood defence measures

As described in previous chapters, the Great Brak estuary is vulnerable to coastal and overland flooding events. Low-lying existing properties adjacent or in the estuarine functional zone have experienced significant flooding in the past. The focus of this study, however, falls on flood protection measures specifically for The Island.

There are four approaches to coastal flood defence measures evident in the literature. These categories are (1) defending the current shoreline, (2) advancing the current shoreline, (3) retreat and (4) a do-nothing approach (USACE 2006). This section will describe several possible flood defence measures that may be applicable to the study area. The estuarine hydrodynamics and coastal flooding components, described in Sections 2.3 and 2.7 respectively, as well as the site-specific characteristics and the expected hydraulic loadings, will be considered in the formulation of possible solutions. The Do-Nothing approach is mostly adequate where the risk of extensive property damage and loss of life is low, which arguably makes it an inadequate solution for the case of the Island.

The potential flood defence measures can be sub-divided into three main categories. The categories are:

1. Management options
2. Hard engineering options
3. Soft engineering options

Alternative flood defence measures, like flood attenuation, will also be explored. The categories and the relevant flood defence measures will be expanded on and discussed in this section.

4.1 Management options

The management flood defence options are mainly aimed at new development in areas where possible flooding may occur. The applicability of these options on existing developments are limited and will be addressed.

4.1.1 Adaptation

The CEM (2006) describes two methods of non-structural adaptation methods for flood prevention, mainly applicable to new developments. The first method is to establish the 100 – year flood line, to determine the risk of flooding. This flood line can then be converted into Flood Insurance Rate Maps. Where developments located above the flood line are charged significantly lower premiums for property insurance, which acts as an incentive for developing above the flood lines. The second method is to establish adequate Setback limits, which will limit construction in the coastal hazard zone (USACE 2006).

4.1.1.1 Flood insurance rate mapping

Insurance companies in South Africa, like Santam Pty (Ltd), are assessing the return period flood lines for coastal hazard zones, which includes estuaries, every 5 years for major rivers. These flood lines are used in their risk assessment practice to determine the level of flood risk involved. Established companies like Santam Pty (Ltd) will decline to quote or suspend cover for flood damage if the risk is too high. However, the insurance market is very competitive which leads to willingness from companies to take on high-risk clients to establish a short-term market share. This inevitably causes a backlash of distrust by the consumer (Denichaud. 2017 pers. comm.), and arguably stunts the effect that Flood Insurance Rate Mapping could have on coastal adaption.

4.1.1.2 Setback line delineation

Setback line determination for development has been made a legal obligation by the Integrated Coastal Management Act (2008) in South Africa. For development in estuarine environments, setback line delineation is currently done by a pre-selected contour line, where in-depth hydrodynamic studies of the specific estuary have not been undertaken, usually the +5 m MSL or 8+ m MSL contour line is chosen. Detailed numerical hydrodynamic studies of the estuary, to determine the 50 or 100-year flood lines, should consider a wide variety of parameters. Topographical surveys, super critical flows, roughness coefficients and the dynamic estuary mouth geometry (width and depth) should be taken into account by individuals specialising in estuary hydrodynamics (Theron. 2016). The lateral boundaries demarcated by proof of tidal and riverine intrusion (like mud or sand flats) wetland areas and beaches should also be an indication of the estuarine functional zone.

For setback line delineation regarding existing development surrounding coastal and estuarine functional zones (areas that may experience flood damage due to ocean and fluvial flooding events), the seaward facing cadastral line is taken as the setback line by the methodology followed by the City of Cape Town (CoCT). This is done to avoid the legal implications which may be associated with the event where the calculated setback lines lie landward of existing private property lines. The set-back lines, however, can influence how existing development is maintained. For developed beach front areas, the CoCT uses hazard overlay zones, calculated from various return period flooding conditions and SLR projections to accompany the set-back lines (Van Weele, *et al*, 2014). This is done for regional town planning control and will inform owners with development rights of the expected risk. See Figure 4-1 for an example of the combination of setback lines and hazard zoning. The low-risk zone refers to the 100 – year horizon, where the medium- and high-risk zones refer to the 50 – and 20 - year horizons respectively.

Figure 4-1: Setback line (black and white) and hazard overlay zoning (Source: Van Weele et al 2014)

Coastal hazard zoning and setback line delineation for the Great Brak estuary will prohibit new developments but will have a small impact on the existing developments. It will inform residents of the long-term risks they face and may entice retreating to areas free of flood risk.

4.1.1.3 Retreat

Based on hazard risk mapping and setback line delineation, the retreat option can be explored. The retreat option entails the acknowledgement of the potential long-term risk estimated for extreme and future flooding related events and ultimately the decision to move infrastructure to an elevation above the calculated flood lines. This option is less applicable to established residential developments, like the Island in the Great Brak estuary, and more applicable to transportation-related infrastructure, i.e. roads, bridges, railway lines; and to important services, i.e. sewer and water lines.

4.1.2 Accommodation

Accommodation of floods (or flood allowance) entails the acceptance of the flood risk and imposing measures that are not direct flood protection measures. Emergency evacuation protocols, raising existing property and more robust buildings and foundations are forms of flood accommodation.

4.1.2.1 Raising of existing property

Property located in the coastal hazard zone may experience extensive flood damage due to inundation of foundations. Private home owners can make the decision to accommodate the expected flooding and

Figure 4-2: Example of a raised house to accommodate flooding (Source: www.allplans.com 2017)

explore the option to raise the existing property to a safe elevation. This option will likely not be possible for all house types, wooden house structures would be more suitable for raising than brickwork structures. A two-story house can perhaps achieve the raising effect if all valuable items and bedrooms are relocated to the second story. See Figure 4-2 for an example of a raised house concept.

4.1.2.2 Emergency evacuation protocols

Emergency evacuation protocols are a form of flood accommodation, and should be based on early warning systems and planned evacuation routes. Evacuation procedures should be pre-determined and all affected parties should be informed and included as part of the planning process. Emergency drills can be held to practice the evacuation procedure to ensure success in the event of an extreme flood event.

The Island, in the Great Brak estuary, only has one entry and exit point, which may exasperate emergency conditions. Accurate early warning systems and communication channels will be essential to warn the residents of extreme flooding and to ensure the safety of the residents.

4.2 Hard engineering

Hard engineering, flood defence options entails the construction of shoreline protection structures. This will normally be a last-resort option, as the economic and environmental implications are considerable. Some conventional hard engineering flood defence options will be discussed in this section.

4.2.1 Dike

A dike is an earthen mound, usually constructed using fine material, like sand and clays to make a high, impermeable structure. The structure generally has a gentle seaward slope and reduces the wave run up and the erodible effect of waves (USACE 2006). The seaward slope of the dike will generally be armoured by means of grass, asphalt concrete slabs or rocks. The primary function of a dike is to protect low-lying property from flooding from the ocean. See Figure 4-3 and Figure 4-4 for example cross-sections of sea dikes.

From the figures can be seen that the gentle slopes needed in the construction of sea dikes make it obvious that a wide area will be needed to achieve the desired height of the structure. For the specific case, of the Island in the Great Brak estuary, the space around the Island is limited and the complex estuarine environment is very sensitive to anthropogenic intervention.

The estuarine environment will evidently deal with fluvial and oceanic induced flooding due to heavy rains in the catchment and to the influence of tides in the estuary. A longitudinal dike can also be used to achieve the desired crest level and to stop the high velocities achieved in the main channel of the river in flood, of inducing scour and inundation to the Island. A longitudinal dike is similar to a sea dike in cross-section, they differ only in the area that they are used. Longitudinal dikes are used in river channels and is located on the river banks parallel to the river channel. The longitudinal dike will likely have to be armoured by rock to protect the structure and a have highly protected toe area, to protect the structure from being undermined (CIRIA 2007).

Figure 4-5 shows a typical cross-section of a longitudinal dike per the Rock Manual (2007). The longitudinal dike is normally used to stabilize a section of river in the horizontal direction, and is to some extent what is desired at the island. A longitudinal dike has steeper sloped areas as it is protected adequately, resulting in a structure less wide than a sea dike for the desired crest level.

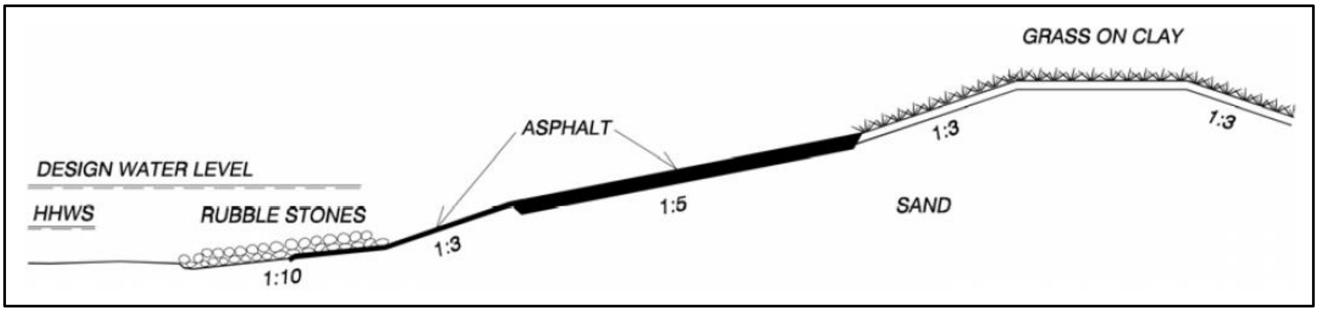


Figure 4-3: Example cross Section of an armoured sea dike (Source: USACE 2006)

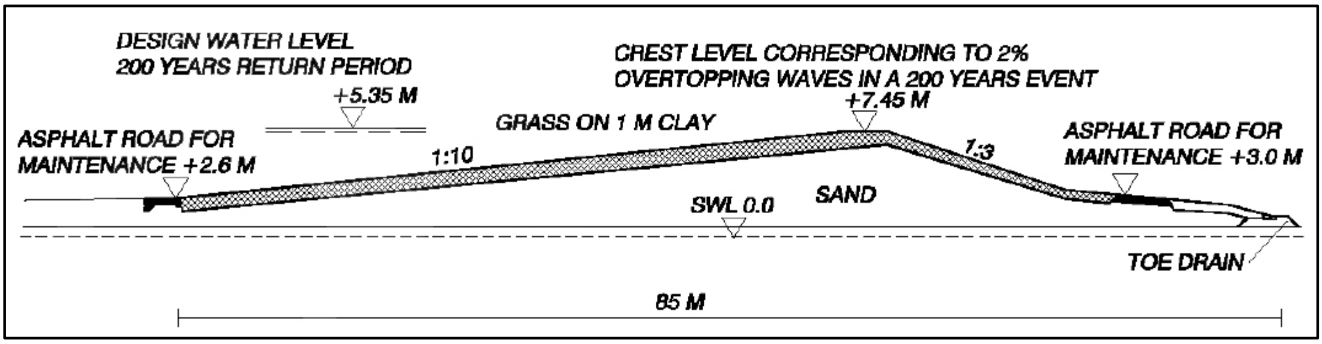


Figure 4-4: Example cross Section of a grass-armoured sea dike (Source: USACE 2006)

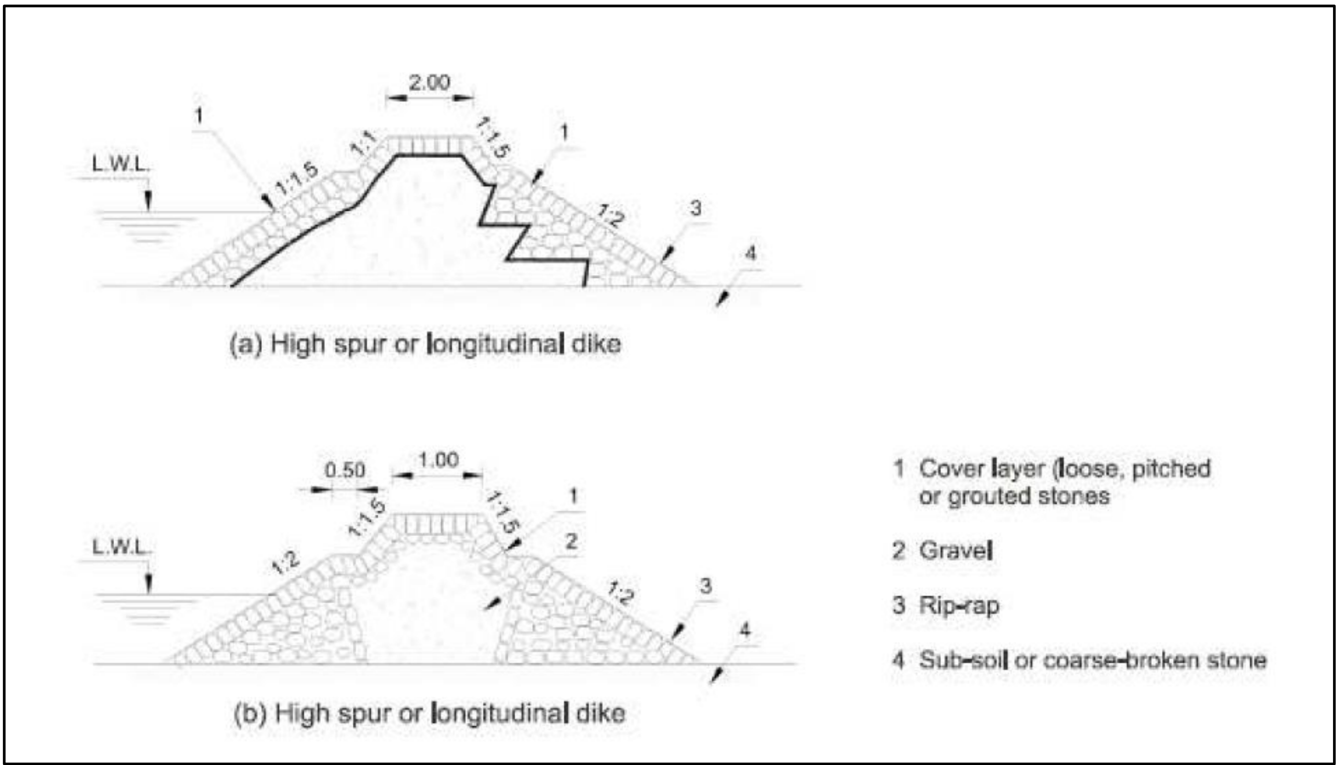


Figure 4-5: Typical cross Sections of a longitudinal dike (Source: (CIRIA 2007)

The crest level of this structure will be influenced mainly by floods, where a probabilistic based approach is needed to determine the expected water levels in the estuary. Other than the design water level derived from the probable floods is freeboard. The freeboard is the required minimum vertical difference between the design water level and the structure crest level. The freeboard can be influenced by the expected settlement of the subsoil used in the structure, the effect of wave run-up in the presence of wave penetration, effects of possible seiches and the expected rise in mean sea level over the design lifetime (CIRIA 2007).

4.2.2 Revetment

A revetment structure to stabilised banks of rivers is some of the most commonly used protection according to the Rock Manual (2007). A revetment structure is a composite structure, comprised of a form of armouring placed on an under-layer of placed material. The under-layer functions as a transition between the larger armour material and the fine erodible materials of the foundation soil and is generally made of crushed rock or gravel (CIRIA 2007). To further protect the finer subsoil, geotextiles may be incorporated to stop the finer material that comprises the foundation from eroding. See Figure 4-6 for an example cross-section of a revetment structure. The nature of the armour material will be governed by the design loadings expected on the structure. Most commonly, armour rock made of quarried or field rocks are used. Some alternatives are concrete blocks or concrete slabs that are cast in place, sand filled bags or gabions (USACE 2006).

Like the dike, the revetment is a rubble mound structure and the armour rock dimensions will be a function of the expected extreme loadings on the structure. The extreme wave conditions and flood velocities will govern these loadings (USACE 2006, CIRIA 2007). The crest level of the structure will be determined by the design flood level plus a freeboard. The toe of the structure is the most important part, as it stops slipping failure of the structure slope, thus the depth of the toe needs to be equal to the maximum expected scour. A steeper slope is allowable with revetment structures – a maximum slope of 1:1.5 (V:H) is prescribed in the literature.

The revetment structure is essentially one half of the rubble mound dike structure except that the crest level of the revetment needs to be equal to the river bank height. This halves the desired width of the

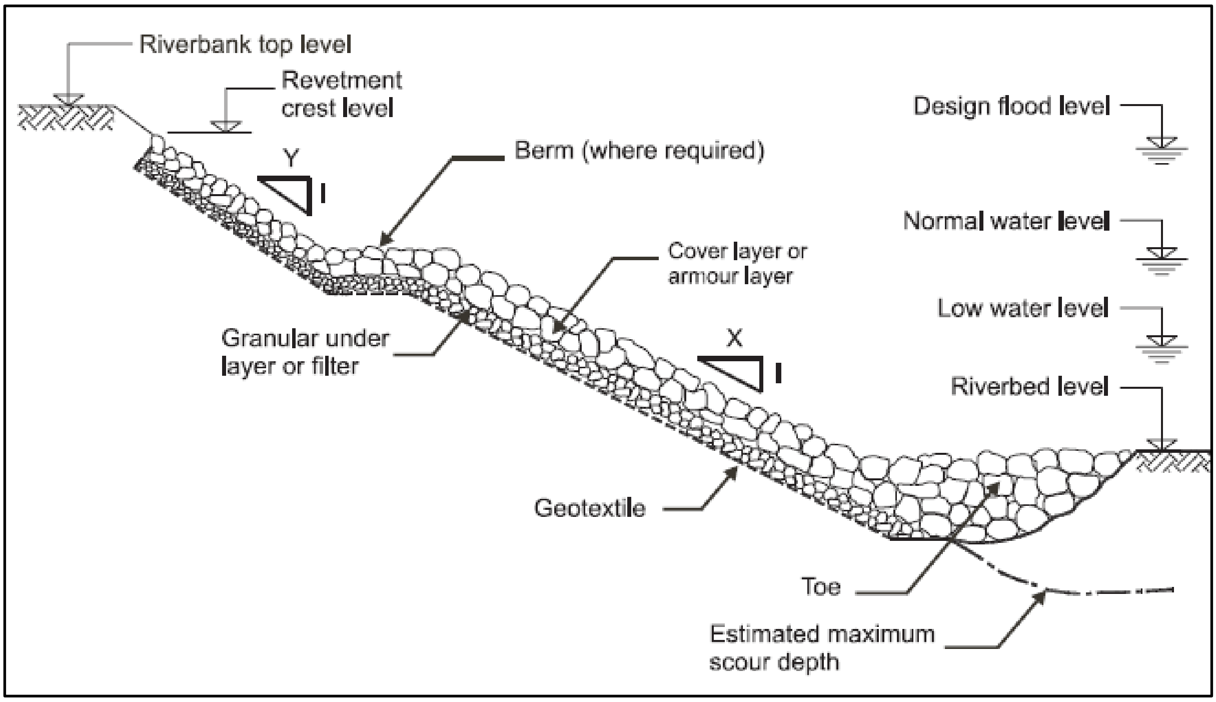


Figure 4-6: Example cross Section of a rock revetment structure (Source: (CIRIA 2007)

structure, but also needs a higher lying bank area behind it to work. This will not be achievable around the whole Island area, as most of the Island is situated below the 5 m MSL contour line.

4.2.3 Sea or retaining walls

A sea wall is a structure placed onshore with the primary objective of stopping coastal erosion and to alleviate flooding of low-lying development (USACE 2006). The sea wall will generally be designed for a wave loading and to account for overtopping and is placed parallel to the shoreline. In terms of river training works, the sea wall translates to a retaining wall. The Rock Manual (2007) prescribes retaining walls as a possible solution where a revetment or dike structure will not be sufficient due to spatial constraints. The key design element of sea walls is the crest height, which is normally designed to stop or minimise wave run-up and overtopping.

A sea wall can have a sloped or vertical seaward profile, a vertical sea wall will be considered, as a sloped sea wall will be identical to a revetment structure – the difference being that sea walls are generally dominantly concrete structures. A vertical sea wall will not dissipate any wave action like sloped structures. See Figure 4-7 for a typical cross-section of a mass concrete sea wall. According to the CEM (2006), scour in front of the sea wall will be enhanced due to wave reflection off the wall, causing the beach slope to become more gradual consequently allowing for larger waves to reach the sea wall. Thus, scour protection should be added to protect the toe of the structure from being undermined.

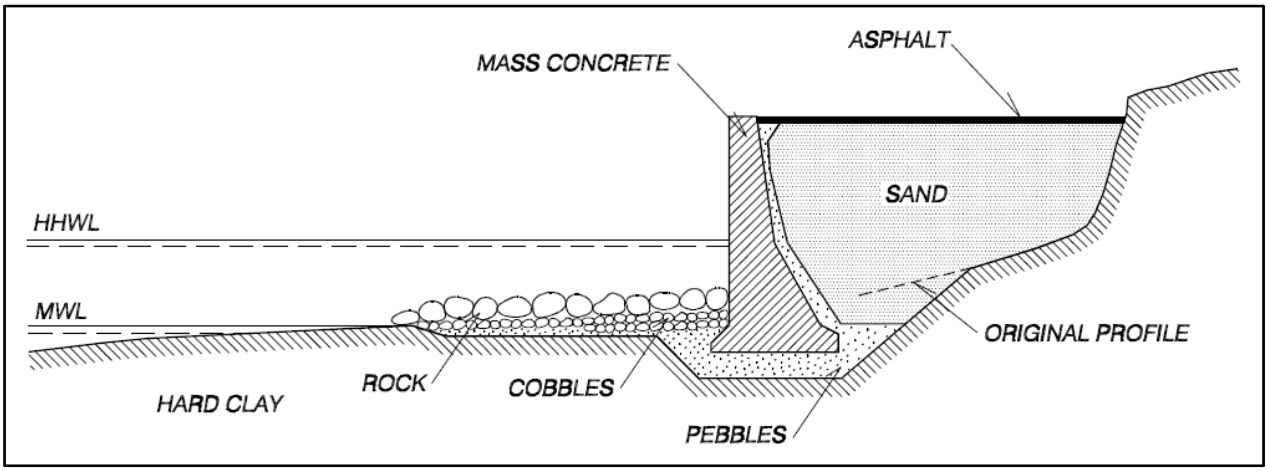


Figure 4-7: A typical cross Section of a mass concrete sea wall structure (Source: USACE 2006)

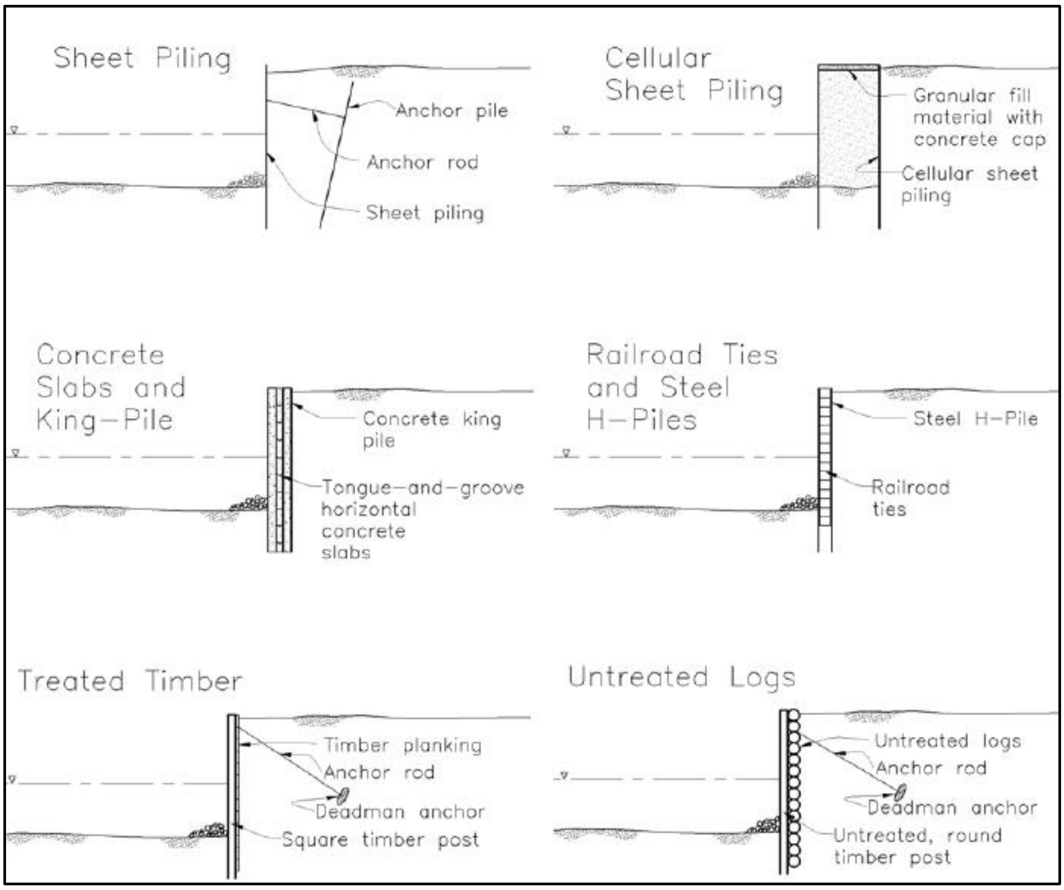


Figure 4-8: Example cross Sections of sheet piling sea wall structures (Source: USACE 2006)

Sea walls can also be constructed using tied walls using steel or concrete piling (USACE 2006). A form of steel piling is the use of sheet piles that are generally placed on bedrock. A concrete capping resting on backfill and the sheet piles can create a boardwalk area on top of the structure. The sheet-pile sea wall will then have a vertical seaward profile. See Figure 4-8 for example cross-sections of piled sea wall structures.

The Rock Manual (2007) suggests the use of gabions to construct a retaining wall. Gabions are steel casings filled with rock. Gabions allow for vegetation growth and free drainage. The casings can be tailor-made to a desirable size on site. The rock size is subject to the design current velocity and the

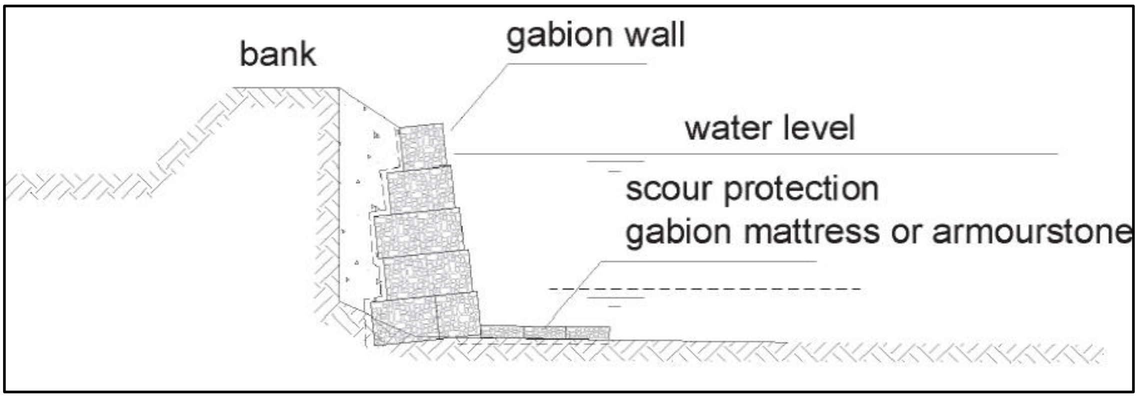


Figure 4-9: Example cross Section of a gabion retaining wall (Source: CIRIA 2007)

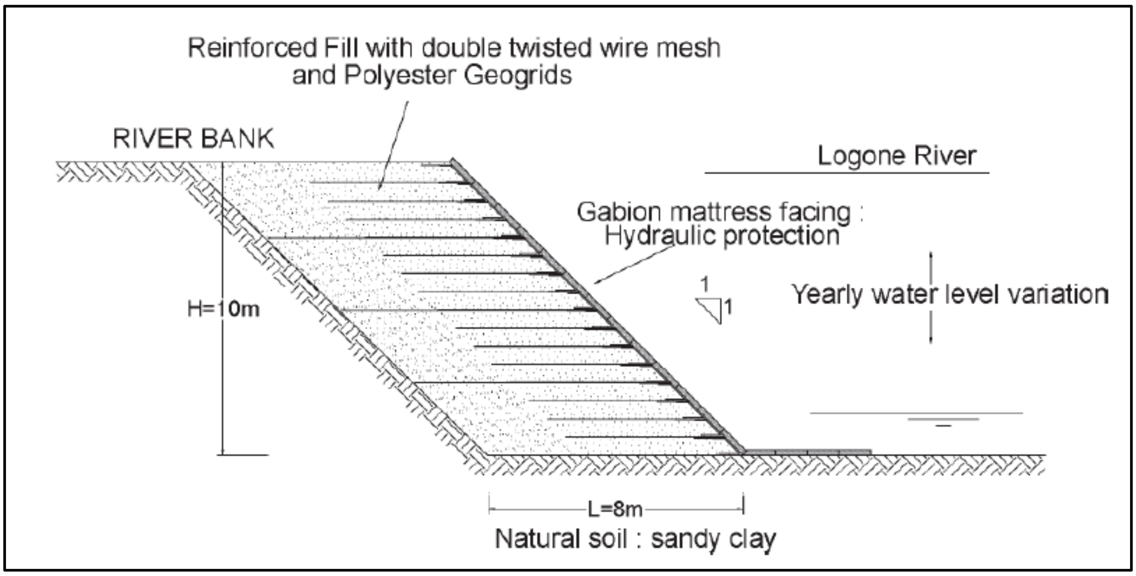


Figure 4-10: Example cross Section for a sloped gabion mattress bank protection (Source: CIRIA 2007)

expected wave conditions. Gabions may be suitable for cases where a maximum current velocity does not exceed 6 m/s and the expected wave does not exceed 1.5 m. From Theron, Barwell *et al* (2012), gabions are to be a temporary solution in high wave energy environments with a lifetime of 3 – 5 years. The lifetime can be significantly increased if used in the backshore area, which experience less frequent wave attack.

The durability of the structure is subject to the durability of the wire casings and the quality of the armour rock placed inside. The steel casings will need a form of corrosive protection – in river environments a zinc or galfan (Al-Zn Alloy) coating is generally applied and in marine environments, where chemical aggressiveness due to the salinity a plastic covering is generally applied (PVC or polyethylene) (CIRIA 2007). The gabion protection can be vertical or sloped – a steeper slope may be achieved with gabion mattresses. See Figure 4-9 for an example of a vertical gabion structure and Figure 4-10 for a sloped gabion structure.

4.2.4 Shore parallel structures

Shore parallel structures are built parallel to the shore, and can be designed to be submerged or above the water. These structures are usually built along shores vulnerable to erosion to stabilise the shoreline. The structures induce wave breaking and limit the wave-induced erosion. Examples of shore parallel structures are detached breakwaters, artificial reefs, rock berms and even tidal pools as multifunctional structures (Theron, *et al*, 2012).

An accretion of sediment can be expected, leading to a wider beach, and ultimately added protection from ocean flooding consequences. In the case of the Great Brak estuary, the overtopping of the beach berm can be reduced due to the structure and resulting wider beach. The shore parallel structure causes wave energy dissipation by inducing diffraction, reflection, shoaling effects and increase bottom friction on passing waves, which allows the beach to build up. The complexity of design increases if a submerged structure is chosen. The most successful designs are the impermeable, water surface piercing structures with a relatively high crest height (+2 to +4 m MSL) structure. This might decrease the visual amenity factor of the beach environment and if designed incorrectly, can cause down drift erosion on adjacent beaches. See Figure 4-11 for an example of a submerged artificial reef structure and the sediment accretion in its lee.

There are advantages and disadvantages to this type of structure. The advantages are mostly related to the protection from aggressive storm wave attack. The estuarine mouth dynamics, however, are very sensitive to sedimentation as a closing mechanism and the wider beach caused by the structure may increase the possibility (and duration) of mouth closure.

Figure 4-11: Example of a submerged artificial reef structure and the accretion of sediment in its lee (Source: AK Theron, L. Barwell, *et al*. 2012)

4.2.5 Perched beach

Perched beach structures function similarly to shore parallel structure. It induces wave energy attenuation through depth-limited wave breaking. A perched beach is essentially a submerged rubble mound breakwater or dike (sill) combined with beach nourishment. The dike functions as a retaining wall for the sand that is placed in its lee, which in turn increases the elevation of the beach (USACE 2006). The dike is essential to this defence option and causes the beach to retain its width. See Figure 4-12 for a description of the perched beach concept.

The perched beach has the advantage of not being visually obtrusive and can be done without affecting the amenity and “sense of place” of the beach area. However, like the shore parallel structures, the increase in beach width will negatively affect the sediment flushing efficiency of river floods through the estuary. The tidal intrusion into the estuary might also be negatively affected.

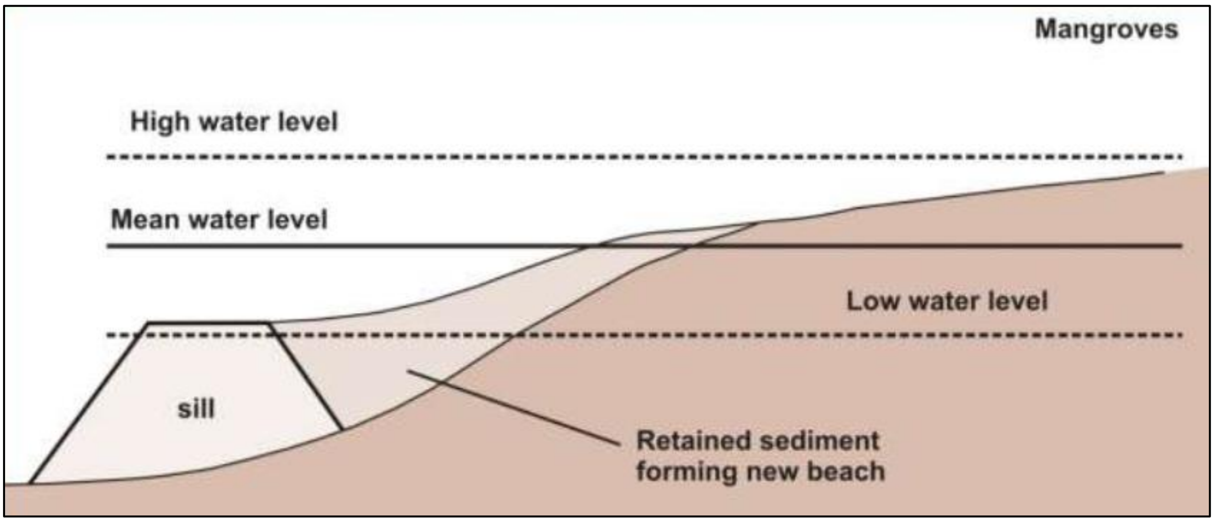


Figure 4-12: Perched beach concept schematisation (Source: AK Theron, L. Barwell, *et al*. 2012)

4.2.6 Tidal inlet stabilisation

Tidal inlet stabilisation is a permanent mouth manipulation, normally done to establish a navigable channel. This mouth stabilisation is normally feasible when the tidal inlet is used as a ship navigation channel for an inlet harbour. The permanent connection to the ocean is normally achieved by using jetties, breakwaters and dredging techniques. See Figure 4-14 and Figure 4-13 for examples of tidal inlet stabilisation using breakwaters and jetties.

Tidal inlet stabilisation will take the variable of the mouth condition and berm height out of the equation and subsequently allow for fluvial floods to run into the ocean through a less constricted mouth, thus resulting in decreased flood levels in the estuary. If a breakwater is incorporated, wave penetration into

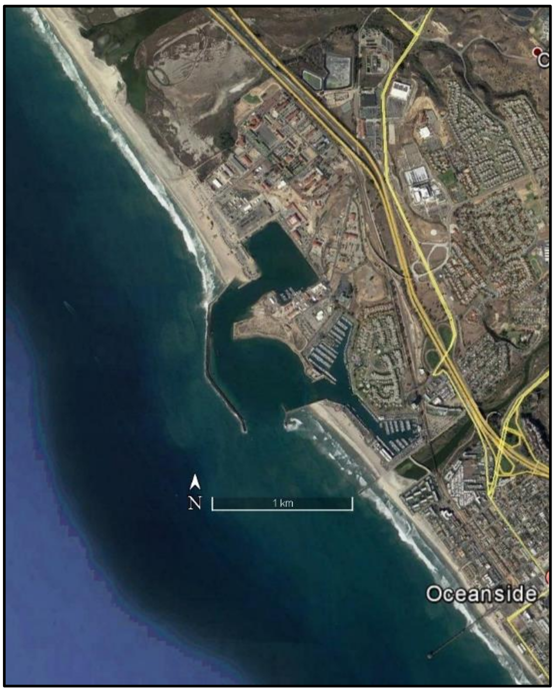


Figure 4-14: Example of a stabilised tidal inlet by using breakwaters in Oceanside, California (Source: Google Earth 2017)

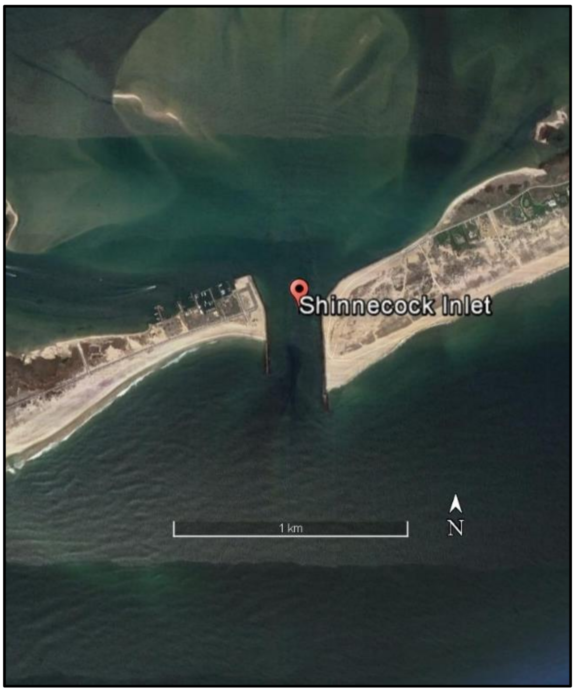


Figure 4-13: Example of tidal inlet stabilisation using jetties at the Shinnecock Inlet, Long Island, New York (Source: Google Earth 2017)

the estuary will be limited. Negative side-effects are generally seen on adjacent beaches as the sediment transport dynamics of the coast is influenced. The estuary will also permanently undergo a change in functionality, ecology and will lose the appealing natural aspect. The applicability of this option to the Great Brak estuary is limited, as the estuary is not used for a vessel harbouring function.

4.3 Soft engineering

Soft engineering can be described as semi-natural interventions for flood proofing of coastal developments. The possible soft engineering options will be discussed in this section.

4.3.1 Dune stabilisation

High ocean waves have, during closed mouth conditions, caused extreme water levels in the estuary (see Section 3.4) due to overtopping of the estuary mouth sand berm. The sand berm has a variable height due to the sediment processes like aeolian sediment transport, erosion and accretion due to high and low river flow periods, as well as the erosion and accretion from large wave events and longshore sediment transport gradients.

A high crest elevation for the sand berm will provide enough protection against overtopping-induced extreme water levels in the estuary basin. A higher crest elevation can be achieved by utilising the natural sediment transport processes at the site to create a dune structure at a preferred location. Two methods to achieve dune stabilisation are described in the CEM (2006). The combined use of the methods is also encouraged. Dune stabilisation can be achieved by using fences and by introducing

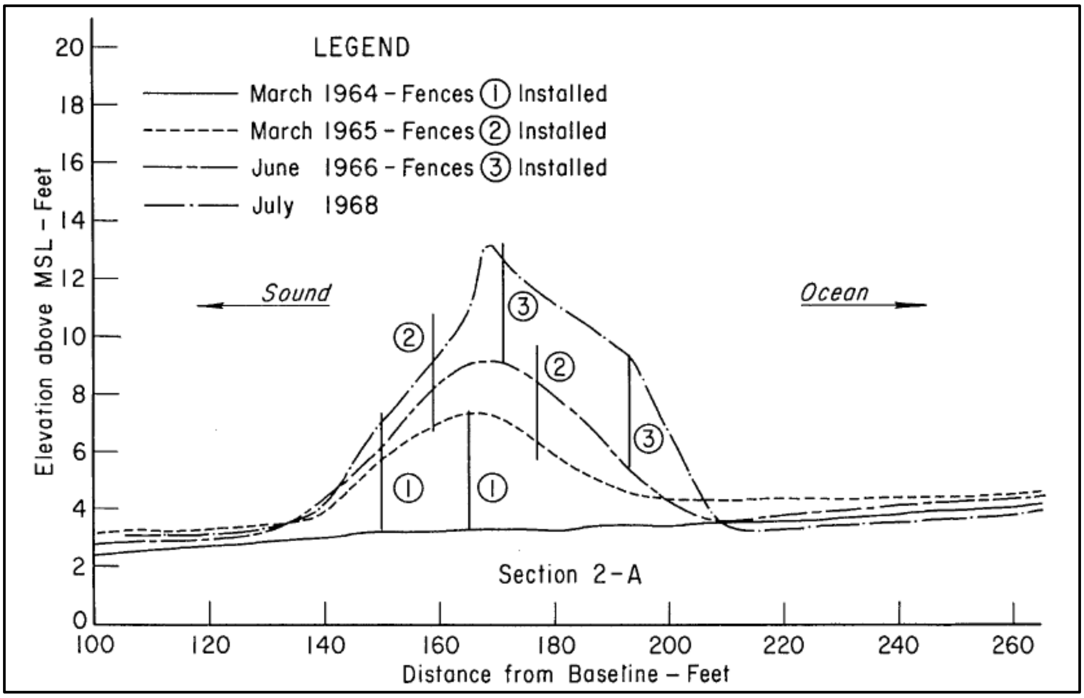


Figure 4-15: Concept of sand accumulation for dune stabilisation using fences (Source: USACE 2006)

vegetation to the preferred site. See Figure 4-15 for an example of sand accretion over time achieved with fences. Dune stabilisation is normally part of beach nourishment design, to establish a healthy sediment reservoir. The stabilisation measures are deemed relatively inexpensive and serve two beneficial purposes. It enhances the protective nature of the dune and serves to reduce sediment losses (USACE 2006). The degree of protection will be dependent on the alongshore length of the dune and the crest height. If a certain crest height is achieved, the dune field will stop overtopping from large ocean events, for the whole length.

The vegetated dune can be armoured by placing gabion boxes or rock protection underneath the dune. During ocean storm events, the dune can be eroded to an extent, but the armouring will prevent excessive erosion from occurring. The dune can then build up naturally or dune replenishment may be needed.

The influence of a stable dune on the estuary mouth will be the concerning issue regarding this option. A certain dune crest level will need to be obtained to defend against overtopping waves, but the estuary inlet channel still needs to connect to the ocean periodically. The width (and depth) of the estuary inlet greatly affects the achievable water level in the estuary during overland flooding events. If the mouth can't be flushed open wide and deep enough, the water will dam up too high in the estuary basin and cause flooding to the low-lying properties. The length of the dune, L_D, will be a crucial design parameter, as it will only stop overtopping for the length of the dune field. It cannot be so long as to

Figure 4-16: Potential site for developing a dune field for natural protection from ocean flooding (Source: Google Earth 2017)

constrict the mouth from opening wide enough and won't help much if it is designed to be too short. See Figure 4-16 for a schematisation of the dune stabilising concept at Great Brak estuary.

The dune only has an impact on ocean induced flooding and has little to no effect on river floods. If the dune constricts the mouth width it may even exacerbate river flood conditions. The dune may cause a sediment build up in the western channel. It may, however, force more river flooding around the eastern side of the Island and cause more scouring of the eastern channel. Sedimentation in the eastern channel is perceived to be a problem at present.

The certainty of the environmental impacts of stabilising the dune field is low. The estuarine mouth is already being manipulated frequently, the added manipulation of a dune field may cause the estuary to close indefinitely due to the increase in sediment availability. The concept may hold merit for investigation, however, the practicality of stabilising the mouth and essentially increasing the beach elevation will need to be assessed using physical and numerical modelling. If an optimal estuary inlet width can be found for a certain design flood, it will determine the maximum length of the dune field. The desired height of the dune field can be calculated by an overtopping analysis of the beach, where the incident wave conditions and the beach geometry will be the controlling factors.

The emergency management protocols of the estuary will remain important to establish a connection to the ocean if a large overland flood is expected. Especially if the beach elevation is increased. The flushing efficiency of the estuary will also need to be investigated.

4.3.2 Beach nourishment

Localised beach nourishment can be done to combat coastal erosion and to establish a wider beach area which will act as a buffer against wave attack, and can in the case of the Great Brak estuary limit the overtopping of the beach berm. Nourishment of the beach is a better shoreline defence option than the conventional hard engineering solutions, but can be as expensive as these solutions (Theron, *et al*, 2012). Eventual re-nourishments will be needed, typically spaced out between 6 to 12 years for an optimal cost/benefit ratio. Nourishment schemes are also used in combination with conventional hard engineering solutions and dune stabilisation. Nourishment of the beach will only help defend against ocean induced flooding and will have no effect on fluvial floods, thus beach nourishment will only be viable in combination with a river flood defence measure.

The increased sediment around the estuary mouth again poses a significant amount of uncertainty and will manipulate the river inlet's frequency of closure. The width of the inlet will decrease and the tidal intrusion will be inhibited. The flushing efficiency of the estuary will also be affected negatively if the beach is too wide and may cause unnecessary damming of water in the lower estuary basin if an emergency breach is attempted.

4.4 Alternative flood protection measures

4.4.1 Flood attenuation

The Wolwedans Dam was completed in 1990 and started filling. The dam's capacity is 25 million m^3, which is equal to two thirds of the Virgin MAR. See Section 3.3.2, for more technical information regarding the dam. The purpose for the dam is solely to provide water to the region and was not built for flood attenuation (Huizinga 2017 pers. comm). From Section 2.5.2, regional climate change models predict a significant increase in flood risk for the catchment K2. It was recommended, by the LTAS Flagship Program (2014), that dams should re-evaluate the dam operating rules to possibly incorporate a form of flood control as an adaptive measure against climate change.

A large reservoir of water will evidently cause an attenuation effect on passing floods, as the inflow and outflow hydrograph for the reservoir will most likely be different. The rate of water flowing into the dam will get dampened by the smaller outflow rate over the dam structure. The attenuation effect of the dam has been considered in most of the hydrological literature for the area. Because the dam was never intended for flood attenuation, the calculations were always done where the dam was assumed to be at the full supply capacity. If the dam is full when a flood flows in, the total volume of water flowing in will eventually pass over the spillway; however, the temporal storage created by the difference in inflow and outflow rates will dampen the peak flows. The attenuation effect calculated in the literature is expressed in percentages of the difference between the peak inflow and peak outflow rates of different return period floods (see Equation 4-1). See Table 4-1 for a summary of calculated dam attenuation percentages from previous studies.

$$\%\ attenuation = \left(1 - \frac{(Q_P)_{outflow}}{(Q_P)_{inflow}}\right) \cdot 100 \qquad \text{4-1}$$

Table 4-1: Percentage of flood attenuation by the Wolwedans Dam

Event	DWA (1990)	Pieterse (2014)	Du Pisani (2015)
Estimated maximum flood	8.7 %		
Regional maximum flood	9.75 %		
1:200 – year flood	12.63 %		6.88 %
1:100 – year flood	13.23 %	10.13 %	8.57 %
1:50 – year flood	14.25 %		9.77 %
1:20 – year flood	17.5 %		
1:10 - year flood	20.63 %		

The values expressed in Table 4-1, are worst-case-scenario estimations. The event of a full dam level coinciding with a large storm in the catchment is likely and should be the planned-for event, as the water in the dam is for water supply and not to attenuate floods. However, in the past, large storms have been attenuated much more than estimated, due to the dam not being at full supply capacity. See Table 4-2 for historical floods after the construction of dam was completed. It becomes apparent that the initial dam level, before the flood, and the volume of water flowing from the storm plays a significant role in the amount of attenuation imposed by the dam. The 1993 and 2007 events both happened when the initial dam level of the dam was on ~ 64 %, however, the 1993 flood was attenuated 100% where the 2007 flood only experienced 36% attenuation. The difference was the total volume of each flood. The volume of the 1993 flood was 11.3 Mm^3 and 44 Mm^3 for the 2007 event (Roux, Rademeyer 2012).

Table 4-2: Historic floods and the attenuation effect of the dam

Event	Initial dam level (%)	Peak inflow (m^3/s)	Peak out (m^3/s)	% attenuation
24/09/1993	63.5 %	437	0	100 %
21/11/1996	100 %	427	323	24.35 %
22/11/2007	65 %	626	404	35.5 %
08/06/2011	87 %	557	339	39.1 %

The Wolwedans Dam can play an important role in acting as a buffer against flooding in the town of Great Brak. However, if the dam is full, other than physically lowering the water level of the dam, the only way to ensure (with certainty) that the dam will have the capacity to impose enough attenuation on the large return period floods in the catchment, will be to lower the outflow capacity of the uncontrolled spillway by shortening the overflow length of the spillway.

It will be shown later in the thesis, that a significant amount of water needs to be released from a full dam to adequately lower the water level to impose even a 20 % attenuation on passing extreme floods. Which makes physically lowering the dam water level for the attenuation effect an inadequate option to consider.

4.4.2 Shortening of the spillway overflow length

Shortening of the overflow crest length is a structural option to alleviate flood conditions in the lower reaches of the estuary. It can be managed and designed to be of minimal impact to the ecological needs of the estuary. The water release policy for the ecological needs of the estuary (discussed in Section 3.2.2) can stay the same, as the alteration to the spillway length will only have an impact when the dam is spilling. The alteration will cause a certain attenuation and translation of passing floods and will cause the upstream water levels to reach heights which will not have been reached in floods with the current spillway capacity.

The Wolwedans Dam is classified as a Category III dam, where the dam is deemed to be Large with a High hazard potential rating due to its high dam wall and close proximity to the town of Great Brak (great potential for loss of life). The maximum spillway discharge was found to be 1920 m³/s from the Stage-discharge curve obtained from DWA (2017). In terms of dam safety, the Safety Evaluation Discharge (SED) is the level pool peak discharge that the spillway system must accommodate without failing. According to the dam safety guidelines set out in SANCOLD (1991), the recommended SED for the Wolwedans Dam should be calculated by means of Equation 4-2.

$$SED = RMF_{\Delta+} = 209 \cdot A^{0.46} \tag{4-2}$$

The SED was calculated as 1912 m³/s, which means the Wolwedans Dam spillway is built according to the safety guidelines. To reduce the outflow capacity will increase the achievable water level in the dam which will exceed the allowable water level for dam stability. This will mean that the dam wall will need to be strengthened on the downstream side (Denys. 2017 pers. comm.). This is normally achieved using concrete to increase the cross-sectional area of the dam wall. To strengthen a dam wall structure that is 70 m high will undoubtedly be extremely expensive.

4.5 Combination solutions

Combinations of different types of flood defence measures are possible, and can help ensure the success of the project. Beach nourishment can be done with the construction of a sea wall beach defence to protect the property in the lee, while also maintaining the beach width. Another combination example is dune stabilisation and beach nourishment, done to create a sediment reserve and to capture windblown sand to keep the sand in the beach system and increase the lifetime of the project (USACE 2006).

A combination of solutions will be considered for the case of Great Brak estuary, to optimise the effectiveness of the flood defence measure. The dike structure can be utilised in conjunction with other hard engineering options, to cut down on construction costs. This is discussed in more detail in Section 6.2.

4.6 Cost

The expected cost of the flood defence option will need to be investigated to be incorporated into the evaluation criteria. A first order estimate will be used in the initial evaluation of the potential flood defence measures. See Table 4-3 for a summary of minimum and maximum cost estimates per meter for various flood defence options. The difference between minimum and maximum values are so large as most of the costs involved are based on site specific characteristics like transportation, accessibility and availability of construction materials (Theron, *et al*, 2012).

Table 4-3: Summary of shoreline defence option estimates (Adapted from Theron, Barwell L., *et al*. 2012)

Description	Approximate minimum costs (excl. Tax) per meter	Approximate maximum costs (excl. Tax) per meter
Beach nourishment @ rate of 300 000 m³/a for 10 years	$4000	$60 000
Beach nourishment maintenance	$400	$7 780
Vegetated dune	$750	$7 200
Gabions – semi sheltered location	$1 100	$23 000
Permeable revetment and walls	$2 300	$24 000
Geotextile sand containers – semi-sheltered location	$1 100	$23 000
Rock groynes	$1 000	$29 200
Rubble mound breakwater: land based	$1 500	$15 100
Rubble mound breakwater: marine based	$2 900	$42 800
Sheet piling – parallel to the shore	$2 700	$36 000

4.7 Performance evaluation criteria

The performance evaluation criteria of flood defence measures in the estuarine environment will be discussed in this section. The cost is an important parameter in selecting an adequate coastal protection scheme, however, it cannot be the only criteria. The performance of each option will be evaluated in terms of its hydraulic function, environmental impacts the option might have on the area and the lifecycle cost of the option. The social impact of the flood defence measure will not be explicitly included in the performance criteria of this study, as the identification of all the social aspects surrounding the flood defence measure to be applied at the Island should culminate from a Public Participation Process where all Interested and Affected Parties should deliver input, which is deemed to be beyond the scope of this study.

From Chapter 3, the EMP is deemed an important guideline when developing adequate flood defence measures for low-lying development in the estuary. The environmental and socio-economic impacts will need to be included into the evaluation criteria, which will need to comply with the objectives set out in the Great Brak EMP and National Environmental Management: Integrated Coastal Management Act (Act No. 24 of 2008. 2009). The criteria for performance evaluation will be discussed below.

4.7.1 Hydraulic performance

The flood defence measure for the Island will need to protect the properties by supplying sufficient protection against high water levels caused by extreme rainfall and extreme coastal events. The expected extreme events for the estuary and for the Island are discussed in Section 2.8.4. The hydraulic functionality of the flood defence measure will be judged by the criteria discussed in Table 4-4.

Table 4-4: Hydraulic performance evaluation criteria

CRITERIA	DESCRIPTION
Protection against extreme flooding in the catchment	○ Sufficient crest height of defence on the Island: the 100-year flood line + freeboard ○ Sufficient protection against river flow velocity in the estuary during fluvial flooding. ○ Local scour protection at toe of structure. ○ Allowances for climate change
Protection against extreme coastal flooding	○ Sufficient crest height of defence on the Island: ▪ Closed mouth conditions: berm overtopping. ▪ Open mouth conditions: tidal intrusion with storm surge component. ▪ Allowances for climate change. ○ Possible direct wave attack ▪ Depth-limited significant extreme wave for open mouth conditions and extreme still water levels. ▪ Tolerable overtopping discharges ▪ Allowances for climate change.
Estuarine sediment flushing efficiency	○ Improve flushing efficiency of the estuary to reduce likelihood of long term nett sand build up in the lower estuary basin ▪ Breach berm at highest possible levels to maintain longer open mouth conditions ▪ No significant sediment accumulation in the mouth area
Sustainability of protection measure	○ Lifetime expectancy ○ Maintenance

4.7.2 Environmental performance

The environmental performance criteria will need to be based on the environmental objectives of the EMP. The environmental performance criteria will also include factors which influences the recreational amenity of the estuary and beach area as well as consideration into the protection of the whole estuary reach. The environmental impacts of the proposed flood defence measure will need to comply with the following criteria:

Table 4-5: Environmental impact evaluation criteria

CRITERIA	DESCRIPTION
Keep estuarine ecological status as close as possible to natural state	○ Lifetime ▪ Not influencing the existing water release policy for the ecological water requirement for the estuary. ▪ Promoting open mouth conditions. ▪ Not causing any form of pollution ▪ Not infringing on the nursery function of the estuary ○ Acceptable construction methods ▪ Minimum destruction of habitat. ▪ Turbidity caused by digging/dredging during construction within acceptable limits
Maintain the potential recreational value of the area, especially during peak season	○ Visual impact ▪ For visitors to the Great Brak estuary/beach area. ▪ For the residents on the Island – to not impede their view of the ocean and estuary. ○ Geophysical impact ▪ Should not change/detrimentally impact socio-economic services provided by the estuarine and beach environment, e.g. recreational activities
Beneficial to the whole community	○ Job creation ○ Flood protection for the whole estuary reach ○ Be beneficial for future generations

4.7.3 Economic performance

The Great Brak estuary is a coastal public property and a popular holiday destination. It is important that the flood defence option economically justifiable. The economic performance of the flood defence measure will be judged based in the initial capital cost and by considering the maintenance cost.

Table 4-6: Socio-economic impact evaluation criteria

CRITERIA	**DESCRIPTION**
Lifecycle cost	○ Capital cost ○ Maintenance cost

4.8 Summary of potential flood defence options

Of all the flood defence measures identified in preceding chapters, only a few are feasible. This section will discuss the feasibility for each identified type of flood defence and ultimately conclude with a summary of the flood defence measures to be evaluated further.

The summary of the potential flood defence measures to be evaluated against the criteria set out in Section 4.7 can be seen in Table 4-7.

4.8.1 Management options

The management related flood defence options discussed in Section 4.1.1, will not be evaluated further. The adaptation measures of insurance flood rate mapping, retreat and setback line delineation is not seen as adequate flood defence measures for this situation as it is less applicable to existing properties. The accommodation flood defence measure of raising existing infrastructure will also not be evaluated further as it is not applicable to all properties on the Island, however, private owners can still investigate the possibility of implementing this measure. An emergency evacuation protocol should be in place for all areas where there is a significant risk of flooding, and will be considered as one of the recommendations irrespective of the flood defence measure chosen in subsequent sections.

The estuary already employs a form of management flood defence option in the form of precautionary breaching to open the estuary mouth when large rainfall events or berm overtopping is foreseen or by keeping the estuary mouth open for longer using flushes and low flow releases. This emergency protocol should be considered when assessing possible flood defence structures for the Island. The protocol, however, offers no protection to the Island from direct wave attack during large wave events. This can be mitigated by incorporating another flood defence measure to protect the Island from large waves.

4.8.2 Hard engineering options

Various hard engineering flood defence options for application in the surf-zone were identified and described. The flood defence measures for application in the surf-zone, like the submerged/unsubmerged breakwater, artificial reef and perched beach structures, offer wave energy dissipation services which might alleviate wave-driven flooding and inundation and to a degree climate change effects on the estuary mouth berm. These structures will also cause sediment accretion due to the lesser wave energy and take up sediment out of the system, which might adversely affect down drift beach areas. These structures are reportedly very expensive and complex to design and place. Finally, these structures will not provide any flood defence for the island from river floods, and could even potentially lead to higher water levels during river floods. For these reasons, the shoreline parallel structures will not be evaluated further.

Tidal inlet stabilisation will cause the estuary to be completely artificial, and thus be directly the opposite of the environmental performance criteria. These FDM are also deemed to be expensive, and due to the size of the estuary and to the fact that the inlet channel is not used for ship navigation, this option will not be evaluated further.

Various hard engineering options considered for application in the lower estuary basin, directly around the Island were also identified. Examples of these structures are a dike, a revetment and sea wall type structures. These structures will be visually intrusive but effective in preventing flooding of the properties on the Island. Out of all the hard engineering options described, these structures were deemed to be the best form of direct flood defence against large river and marine driven floods and will be evaluated further in this study.

Altering the spillway dimensions of the Wolwedans Dam to cause more attenuation to fluvial floods is deemed to be too expensive to evaluate further, as any altering of the spillway will result in the water level achievable in the dam to exceed the maximum water level the dam structure was built for. This will affect the stability of the dam and in order to ensure the safety of the residents of the downstream town, Great Brak, the dam wall will need to be strengthened with concrete on the downstream side.

4.8.3 Soft engineering options

The softer engineering options in the littoral active zone, like beach nourishment and vegetated dune are more attractive options than the hard engineering options discussed in preceding paragraphs. However, beach nourishment is more applicable to areas where beach erosion is a problem, the sediment budget at the mouth of the Great Brak River is deemed to be healthy.

The vegetated dune option, where a dune field is established to harness aeolian sediment transport processes to build up a dune will be the only soft engineering option to be evaluated further. The FDM is considered an aesthetically pleasing and relatively cheap option that will keep the estuary at the natural state. The vegetated dune option will defend the estuary mouth berm against “creeping” inlands due to rising sea levels and increases in storminess. The increased berm height will significantly reduce overtopping of the berm from large storm waves and help dissipate large waves. The concern around this option, however, is the influence that the vegetated dune field will have on mouth condition frequency and flushing efficiency during flood events. A vegetated dune on the estuary mouth berm will cause a form of mouth stabilisation and can potentially exacerbate river flood conditions. An integrated combination of a vegetated dune and the current emergency protocol can, if designed correctly, be the most environmentally friendly, socially accepted and economically justifiable flood defence measure, and will thus be evaluated further. The vegetated dune option will be assessed as a stand-alone option and in conjunction with the current emergency protocol.

Table 4-7: Summary of the flood defence options to be evaluated

Applicable area	Option
Lower estuary basin *– potential flood defence measures for application directly around the vulnerable Island perimeter*	**Hard engineering options** ○ Dike ▪ Unarmoured (earthen) ▪ Armoured ○ Revetment ▪ Rubble mound – porous ▪ Concrete – non-porous ▪ Gabions - porous ○ Sea wall ▪ Vertical – non-porous • L – shaped gravity concrete • Mass gravity concrete ▪ Vertical – porous (Gabions)
On the beach – *potential flood defence structures to be placed in the surf-zone to limit the size of the extreme storm waves*	**Soft engineering options** ○ Vegetated dune
Do nothing option	○ Will be considered as the baseline option to compare other solutions with

A combination of potential flood defence options will also be subject to the evaluation and is summarised in Table 4-8.

Table 4-8: Summary of combination flood defence options to be evaluated

Applicable area	**Option**
Lower estuary basin	○ Combination of either: ▪ Sea wall and dike ▪ Dike and Revetment ▪ Sea wall and Revetment The combination structure to be evaluated will be comprised of the two structures that scores highest in the Multi-criteria analysis.

On the beach	○ Vegetated dune and current emergency protocol

5. Design Conditions

The design conditions that a potential flood defence measure at the Great Brak Island must endure will be the focus of this section. The design criteria for river floods will be assessed by means of standard hydrological calculations and for marine driven floods the design storm wave height in depth-limited conditions will be calculated alongside values for storm induced extreme still water levels.

The design life and the chosen return period storm will be the two key parameters in quantifying the design criteria for the potential flood defence measure. The proposed structure will be designed for the 1:1, 1:25, 1:50 and 1:100 return period storm conditions. The design lifetimes under consideration will be dictated by predictions of climate change. Climate change impacts will be included as a vertical SLR component. Predictions for SLR have been discussed in Section 2.5, and the values predicted for the years 2030, 2050 and 2100 will be used to derive the design conditions for that period, dictating the design life of the proposed structure. The design lifetimes under consideration are 13, 33 and 83 years.

The influence the initial dam level has on passing flood peaks will be investigated using a dam basin model, to investigate the alternative flood defence measure of using the dam to achieve a desired attenuation effect on passing floods, by lowering the dam level prior to an expected flood. An overtopping analysis for the estuary mouth berm will be done to investigate the validity of a vegetated dune concept as partial protection against large wave events that will normally overtop the mouth berm.

5.1 Catchment hydrology

The extreme floods estimated for the full catchment area, thus the extreme floods expected to flow into the lower reaches of the estuary will be calculated in this section. As discussed in previous sections, the Wolwedans Dam causes an attenuation effect on floods, and its effect will be incorporated using flood routing techniques discussed in Section 2.6.2. To calculate the routing ability of the dam structure, the catchment areas upstream and downstream of the dam will be dealt with separately, as the routing calculations will only apply to the catchment upstream of the dam, i.e. Area A.

The Mean Annual Precipitation (MAP) for the catchment area recommended by Roux and Rademeyer (2012), will be used as input parameter for Design Flood Estimation, refer to Table 5-1.

Table 5-1: Mean Annual Precipitation (Source: Roux, Rademeyer 2012)

MAP	**730 mm**

5.1.1 Design flood estimation

As discussed in Section 2.6.1, the DRH method was used to determine hydrographs for various return period floods that can be expected in the lower reaches of the estuary. The method was applied to both sub-quaternary catchment areas, Area A and B. The flood routing calculations were done for the current dam release policy – where no water is made available annually for flood attenuation, thus a full dam condition was therefore assumed. See Figure 5-2 for the combined Area A and B extreme flood hydrographs, thus representing the extreme flood hydrographs that will flow into the lower reaches of the estuary.

A 100-year flood event will cause an 865 m³/s flood peak in the estuary and a 682 m³/s flood peak can be expected for a 50 – year flood in the estuary if the Wolwedans Dam is assumed to be full before the flood commences and the area downstream of the dam contributes fully to the run-off.

The input parameters and the hydrographs used in the calculation of the total flood hydrograph (Figure 5-2) can be viewed in Appendix C. From Figure C-2 and Figure C-1 can be seen that at Full Supply Capacity (FSC), the dam attenuates the given return period floods, from rainfall events in the upper reaches of the catchment, by 8.5% and 9% respectively. Of course, this is the worst-case scenario event.

The Design Flood Estimation for Area A compares well with the flood peaks recommended by Roux and Rademeyer (2012) (see Table 3-7) before routing. After routing, the combined Area A and B hydrographs for the extreme floods compare well to the flood peaks calculated by DWA (1990) (see Table 3-5). The calculated hydrographs will therefore be adopted as the Design Flood Hydrographs for

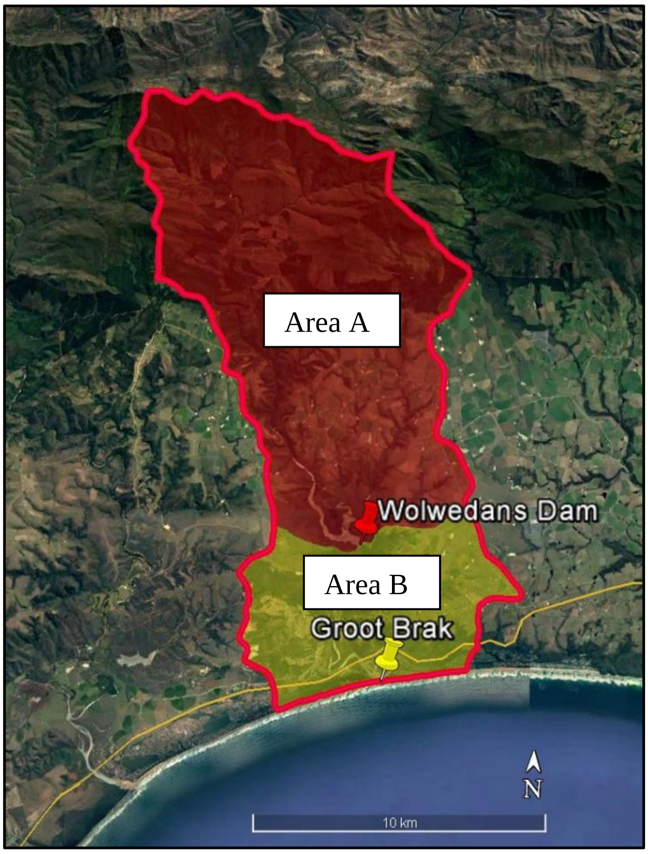


Figure 5-1: Area A (Red) and Area B (Yellow) (Source: Google Earth 2017)

this study and subsequently be used to assess the effect that the initial dam level will have on passing extreme floods.

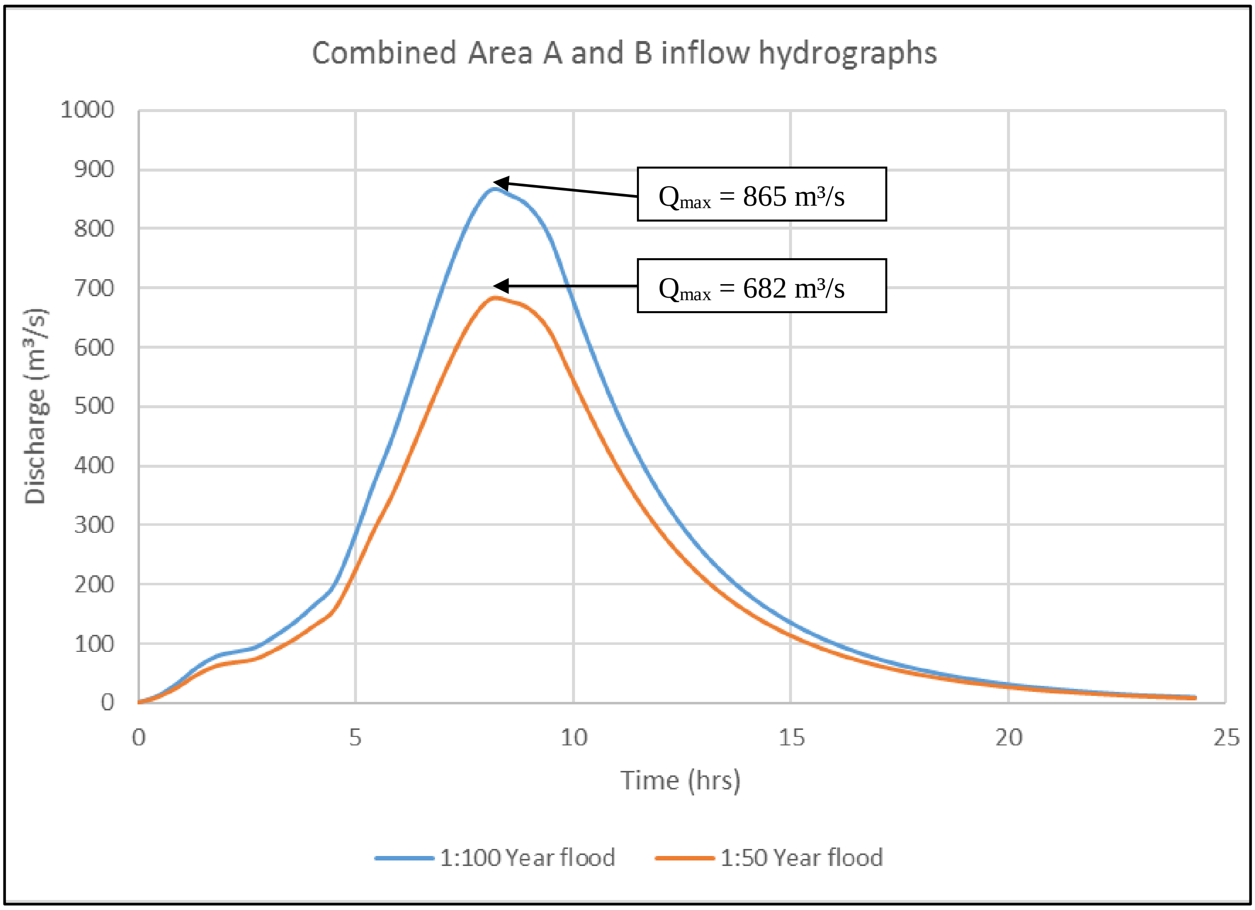


Figure 5-2: Extreme floods estimated to flow into the Great Brak estuary

5.1.2 Effect of initial dam level on flood attenuation

The routing calculations done to determine the attenuation capabilities of the Wolwedans Dam in Section 5.1.1 were done for a 100% full dam. From Section 4.4.1, it is evident that the initial dam level plays a significant role in attenuating floods from the upper catchment area. The routing calculations were adapted to accommodate a variable initial dam level. The percentage of attenuation was calculated, from Equation 4-1, for various dam levels (See Figure 5-3). See Appendix D for the discussion of the dam basin model setup and validation using two recorded floods.

Figure 5-3 shows the effect of a less than full dam on passing extreme floods. The 100 – year flood will experience 100% attenuation if the dam level is at 40% before the commencement of the flood, the same will happen for the 50 – year flood if the dam level is at 45% initially. This relationship between the initial dam level and expected attenuation can be used to assess the merit of lowering the dam level prior to a forecasted flood as a flood control measure for fluvial floods.

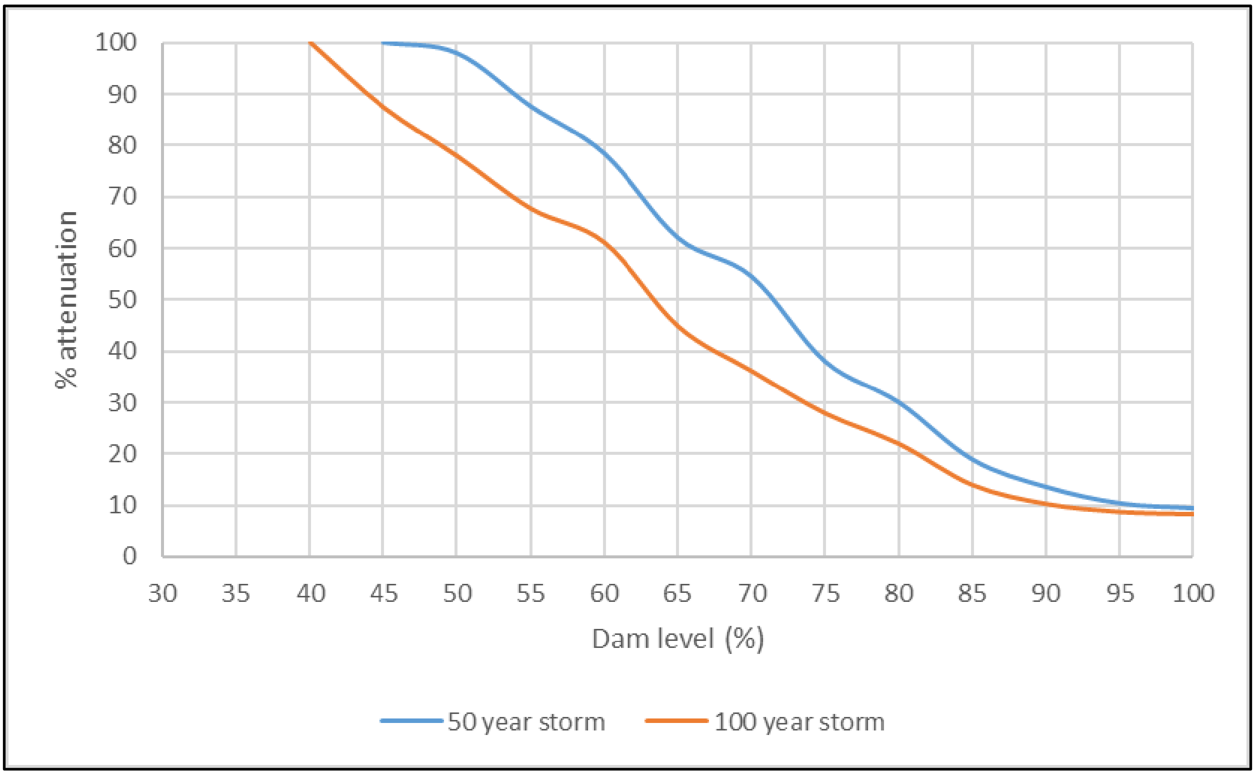


Figure 5-3: Percentage attenuation for different initial dam levels, for the 50 and 100-year occurrence floods for the Wolwedans Dam

Table 5-2: Effect of lowering the dam level prior to the 50 – and 100 – year floods and the volume of water to be released to achieve a certain degree of attenuation

				Total attenuation after release (%)	
Initial Dam Level (%)	Release to (%)	Volume release (m³)	Cumulative release volume (m³)	50-year flood	100-year flood
100	95	1 254 765	1 254 765	10	9
95	90	1 254 765	2 509 530	14	10
90	85	1 254 765	3 764 295	19	14
85	80	1 254 765	5 019 060	30	22
80	75	1 254 765	6 273 825	38	28
75	70	1 254 765	7 528 589	55	36
70	65	1 254 765	8 783 354	62	45
65	60	1 254 765	10 038 119	78	61
60	55	1 254 765	11 292 884	88	68
55	50	1 254 765	12 547 649	98	78
50	45	1 254 765	13 802 414	100	87
45	40	1 254 765	15 057 179	0	100

Table 5-2 shows the calculated required volume of water to release from the dam to ensure a certain degree of attenuation on the 50- and 100-year flood peaks. It has been shown that if the dam is full, the floods will experience less than 10% attenuation. From Figure 5-3 and Table 5-2, the volume of water required to be released can be calculated to ensure a certain amount of theoretical attenuation.

If the dam is full, to ensure 20 % attenuation, a 3.7 million m³ release must be made prior to a 50-year flood and a 5 million m³ release prior to a 100-year flood. These releases will help to reduce the 50- and 100-year flood peaks over the dam wall to 429 m³/s and 535 m³/s respectively, which corresponds to 20-year and 50-year floods (Table 3-7). In an arid country like South Africa, releasing these amounts of water from storage is not deemed feasible, especially with the uncertainties involved in the prediction tools and when the same effect can possibly be achieved by breaching the estuary mouth as a preventative safety measure. The Water Release Policy of the Wolwedans Dam only stipulates 5 million m³ of water for estuary management in a year where the dam level is constantly over 90%, thus, to release the annual budget of water to lower the flood peak for an expected flood is not deemed to be a sustainable and viable solution.

When the dam is at 70% or less capacity, the minimum water volume of 1 million m³ is allocated to the management of the estuary and is normally used to perform the planned breaches of the mouth, thus a preventative breach release prior to a forecasted flooding event is less likely, which might lead to flooding of low-lying properties in and around the estuary even though the flood peak of the 50- and 100-year floods can be seen to be attenuated by approximately 57 % and 36 % when the dam is at 70%. An emergency channel can still be established, if the mouth is closed, which will improve flushing efficiency and help prevent extensive flooding of low-lying properties.

5.2 Wave climate

5.2.1 Extreme wave conditions for Mossel Bay

5.2.1.1 Nearshore extreme waves

As discussed in Section 3.8, Clarke (2016) used wave data captured by the CSIR Datawell Waverider Buoy, located in 24 meters of water and approximately 2.7 km off the coast to determine the extreme nearshore wave conditions for different return period events, by the means of an Extreme Value Analysis (EVA). See Table 5-3 for the nearshore wave conditions for the return periods 1:1, 1:25, 1:50 and 1:100 as calculated by Clarke (2016).

Table 5-3: Nearshore extreme wave conditions (Source: (Clarke. 2016))

Return period event	H_S (m)	T_p (s)
1:1	4	11.9
1:25	5.5	13.1
1:50	5.8	13.3
1:100	6.1	13.6

Given the close proximity of the Great Brak estuary to the CSIR Datawell buoy, the nearshore conditions were accepted as the best estimate and of good quality. These extreme conditions will subsequently be used to determine the significant nearshore wave height, H_S, and the wave run-up level at the mouth of the estuary using the Nielsen and Hanslow (1991) model. See Figure 5-4 for the locality of the buoy relative to the Great Brak estuary. The site, however, is significantly more exposed than the location of the buoy. Alternatively, the updated metocean conditions for the LNG terminal off Mossel Bay (Council for Scientific and Industrial Research, 2009) could potentially be used for better representation of the extreme waves conditions at Great Brak.

Figure 5-4: Proximity of the CSIR datawell buoy and Great Brak river mouth (Source: Google Earth 2017)

5.2.1.2 *Equivalent deep-water wave height*

The equivalent deep-water wave height, discussed in Section 2.7.4, is needed for the use of the Nielsen and Hanslow (1991) run-up model. The equivalent deep-water wave height is obtained by applying an "inverted" shoaling coefficient to the measured wave height at approximately 20 m depth. The shoaling coefficient, K_S, was calculated using Equation 5-2. See Table 5-4 for the calculated equivalent deep-water wave heights for different return periods which will be used in the calculation of the Nielsen and Hanslow (1991) run-up values.

Table 5-4: Equivalent deep-water wave heights for different return periods

Return period event	H_S (m)
1:1	5.29
1:25	7.10
1:50	7.84
1:100	8.28

5.2.1.3 *Deep-water extreme waves*

As discussed in 3.8, the deep-water extreme waves to be adopted in this study were determined by Rossouw (1989) and by the CSIR (von St. Ange. 2017 pers. comm.). The values determined for the Sedco K and FA site recording stations, located approximately 120 km South of Mossel Bay and in deep-water (100 m) were deemed to be most relevant for this study. The deep-water extreme waves will be used to calculate run-up levels using the Mather *et al* (2011) model and to estimate the wave setup using the Karsten (2008) formulation. See Table 5-5 for the adopted values of the offshore significant wave heights and Figure 5-5 for the locality of the FA site and Sedco K recording stations relative to Mossel Bay. The 50% exceedance peak period, T_P= 12.5 s, calculated at the Sedco K station, will be used as the design deep-water wave period (Rossouw. 1989)

Figure 5-5: Proximity of the Sedco K and FA platform and Mossel Bay (Source: Google Earth 2017)

Table 5-5: Offshore extreme wave conditions

(Source: Rossouw 1989 and von St. Ange 2017)

Return period event	H_{m0} (m) – Rossouw (1989)	H_{m0} (m) – CSIR (2017)
1:1	8.31	8.7
1:10	10.23	10.7
1:20	-	11.3
1:50	-	12.0
1:100	12.15	12.6

5.2.2 Significant wave height

5.2.2.1 *Wave energy decay in the surf-zone*

There are a few ways to determine a significant wave height at a site, at different depths, given the deep-water wave conditions. Normally, for detailed design purposes, a numerical model software package, like Delft-3D WAVE (SWAN), is used to determine design-significant wave height at a site, H_S. This falls outside the scope of this study, thus conventional hand calculations will be used to determine H_S, at varying depths.

Nearshore wave transformation processes on open coast areas from deep-water conditions include the refraction and shoaling of the wave profile and ultimately wave breaking. Energy dissipation of waves due to bottom friction is less important than the previously mentioned processes and its effects will be neglected (CIRIA 2007). Refraction is the dissipation of wave energy due to the interaction of an oblique incident wave angle and the bathymetry nearshore. Therefore, the conservative case will be zero refraction and wave crests parallel to the shoreline. For that reason, refraction effects will be neglected for this study. Wave shoaling and breaking will be the only nearshore transformation processes considered. Wave shoaling is the process where a change in wave height can be observed due to waves propagating in varying water depths.

Shoaling of waves can be described in terms of the shoaling coefficient, K_S, which is by definition a ratio of the local wave height, H, compared to the deep-water wave conditions, H_0. The shoaling coefficient can be expressed as a function of water depth, *h*, for a given wave height and period if linear wave theory is assumed. See Equation 5-1 for the mathematical expression for calculating K_S (CIRIA 2007).

$$K_S = \left[\tanh(h)\left(1 + \frac{2h}{\sinh(2h)}\right)\right]^{-\frac{1}{2}} \qquad \textbf{5-1}$$

The shoaling coefficient can also be expressed in terms of the deep-water group wave celerity, C_{g0}, and the local group wave celerity, C_{g1}. See Equation 5-2 for the mathematical expression for calculating K_S (USACE 2006). This expression will be used to calculate the shoaling of the extreme wave for different depths using the methods described in the CEM.

$$K_S = \left(\frac{C_{g0}}{C_{g1}}\right)^{\frac{1}{2}} \qquad \textbf{5-2}$$

Wave shoaling is only important in the shoaling-zone, once the wave reaches the surf-zone, wave breaking occurs which will cause a drop in significant wave height. Two models used in the assessment of wave height decay due to breaking will be described in this section and subsequently used to determine the wave height in the surf-zone (CIRIA 2007).

The Rock Manual (2007) describes two main methods assessing wave decay in the surf-zone. These models are the (1) Van der Meer (1990) model and the (2) Goda (2000) model.

1. **Van der Meer (1990)**

The Van der Meer (1990) method is based on results of a one-dimensional energy decay numerical model which accounts for wave breaking. The graphs shown in Figure C-5, Appendix C, are used to determine the depth-limited significant wave height. The resulting significant wave is obtained as the spectral significant wave height (H_{m0}) which must be converted to the significant wave height (H_S) by the method proposed by Battjes and Groenendijk (2000). The input parameters for the use of the method are the local depth (h), foreshore slope (m_F), deep-water significant wave height (H_{S0}), -wave length (L_0) and the relative deep-water wave steepness parameter (S_{op}).

2. **Goda (2000)**

Goda (2000) developed formulae to calculate the significant wave height in the surf-zone. These formulae are described in Equations 5-3 to 5-6. The Rock Manual (2007), however warns that the application of the model should be done with caution, as it overestimates values by more than 10% for certain cases.

$$H_S = H_{1/3} = \begin{cases} K_S H'_0 \quad for\ {}^{h}/_{L_0} > 0.2 \\ min\{(\beta_0 H'_0 + \beta_1 h), (\beta_{max} H'_0), (K_S H'_0)\}\ for\ {}^{h}/_{L_0} < 0.2 \end{cases} \tag{5-3}$$

Where h and L_0 as defined in (1) and:

$$\beta_0 = 0.028 \left(\frac{H'_0}{L_0}\right)^{-0.38} exp(20m^{1.5}) \tag{5-4}$$

$$\beta_1 = 0.52\,\exp(4.2m) \tag{5-5}$$

$$\beta_{max} = max\left\{0.92, 0.32\left(\frac{H'_0}{L_0}\right)^{-0.29} \exp(2.4m)\right\} \tag{5-6}$$

And K_S like defined in Equation 5-1. H'_0 = the equivalent deep-water wave height.

5.2.2.2 Assumptions

To obtain the design wave conditions at the Island, some simplifying assumptions were made to use the wave-decay models discussed in Section 5.2.2.1.

- The extreme waves calculated by Clarke (2016) from data from the Waverider Bouy (in 24 m depth) represents the equivalent deep-water wave conditions.
- The foreshore slope is uniform and constant.
- The wave angle inshore is normal to the shore i.e. wave front is parallel to the shore.

The assumption to use the nearshore waves calculated by Clarke (2016) instead of the deep-water conditions, was made as Great Brak is located in a relatively sheltered large embayment, and the nearshore recorded waves were deemed to be a better representative of the wave conditions inside the bay. These assumptions proved to be valid as Clarke (2016) observed a good correlation between the conventional hand calculation, of Goda (2000) and Van der Meer (2000), and the results of a numerical wave transformation software package (Deflt3D WAVE) using the inshore extreme wave conditions in Table 5-3 as input parameters. The Goda (2000) and Van der Meer (1990) energy decay models were used to verify the results obtained for the design wave at the Mossel Bay harbour breakwater, 15 km SW of Great Brak. See Table 5-6 for the results of the three methods as calculated by Clarke (2016) at the breakwater of the Mossel Bay harbour.

Table 5-6: Correlation between methods (Source: Clarke 2016)

Method	**Return period**			
	1	**25**	**50**	**100**
H_s – DEFT3D WAVE	3.06	3.28	3.31	3.34
H_s – Van der Meer (1990)	2.96	3.17	3.22	3.24
H_s- Goda (2000)	3.73	3.89	3.92	3.96

5.2.2.3 Depth-limited significant wave height

Waves are deemed to reach the Island during a storm event if the storm surge pushes into the tidal inlet. The local wave height is a function of the water depth, which will be influenced by the SWL (addressed in Section 5.3). The depth of water directly in front of the proposed flood defence structure at the Island will dictate the design wave height.

The depth-limited significant wave height will be assessed for depths of 6 m and shallower, via the one-dimensional energy decay models discussed in Section 5.2.2.1. Later in this section it will be shown that the extreme still water levels deemed possible in Mossel Bay to be +5.65 m MSL, in the surf-zone, and +3.13 m MSL deemed possible for intrusion into the estuary mouth. See Table 5-7 for the significant wave heights at various depths and return periods.

Table 5-7: Depth-limited significant wave height (m) at various depths (m) and return periods (years)

	Goda (2000)				Van der Meer (1990)			
Depth (m)	**1**	**25**	**50**	**100**	**1**	**25**	**50**	**100**
6	3.76	3.92	3.96	3.99	3.40	3.28	3.31	3.29
5	3.22	3.39	3.42	3.45	2.59	2.80	2.82	2.78
4	2.68	2.85	2.88	2.91	2.16	2.22	2.36	2.44
3	2.14	2.31	2.34	2.37	1.68	1.8	1.84	1.91
2	1.60	1.77	1.80	1.83				
1	1.06	1.23	1.26	1.29				

From Table 5-7 it can be seen that the Van der Meer (1990) results are not adequate for depths as shallow as 2 m or 1 m, for the specific relative depths (h/L_0) and wave steepness parameter (S_{op}). It is also apparent that the wave heights calculated by the Goda (2000) formulation are significantly higher than the wave height calculated by the Van der Meer (1990) method. An overestimation of depth-limited significant wave height calculated by Goda (2000), was expected (CIRIA 2007). A specific quantification for the specific case is never discussed, it is merely stated that an approximately 10% overestimation is possible in the nearshore zone. After a 15% adjustment to the Goda (2000) values, better comparison between the two models are observed and will therefore be adopted in this study to assess the wave in depth-limited conditions of 6 m and shallower. See Table 5-8 for the final values for the depth-limited significant wave height. Through means of interpolation, the return period depth-limited wave heights can be calculated at any depth between 6 m and 1 m.

Table 5-8: Adjusted Goda (2000) depth-limited significant wave height (m) at various depths (m) and return periods (years)

	Adjusted Goda (2000)				Van der Meer (1991)			
Depth (m)	**1**	**25**	**50**	**100**	**1**	**25**	**50**	**100**
6	3.20	3.34	3.36	3.39	3.40	3.28	3.31	3.29
5	2.74	2.88	2.90	2.93	2.59	2.8	2.82	2.78
4	2.28	2.42	2.44	2.47	2.16	2.22	2.36	2.44
3	1.82	1.96	1.99	2.02	1.68	1.8	1.84	1.91
2	1.36	1.50	1.53	1.56				
1	0.90	1.04	1.07	1.10				

5.3 Extreme still water levels due to marine hydrodynamics

In Section 2.7 all components relevant to extreme SWL occurrences around the South African coast were defined. The method of superimposing extreme water level differentials for different independent and arguably dependent ocean events is a rudimentary method of assessing the joint probability of occurrence and has been deemed as the best first order estimate of extreme SWL in a data-poor environment like South Africa (Theron. 2016).

The probability of an extreme storm coinciding with a spring tide event (every 14 days) is deemed high enough to be chosen as the design tidal level. Storm surge for these extreme events will then be calculated by its various components: wave-, wind setup and inverse barometric setup. Wave setup was calculated using the methods described in Section 2.7.2. Inverse barometric and wind setup levels were obtained from Theron (2016).

Theron (2016) did an analysis on extreme sea level recordings in order to determine return period residual water levels (excluding tides). These residuals contain the effects of inverse barometric setup, the occurrence of astronomical tides and some wind effects. The effect of wind setup will however not be accounted for separately, as the recorded water level used by Clarke (2016) to determine the deep-water wave conditions also included wind effects. Together with the calculated residuals by Theron (2016), the wind setup will be adequately covered. The residual water levels calculated by Theron (2016) for the Mossel Bay area can be seen in Table 5-9. It should be noted that the values (residuals) are not expressed as an elevation, rather as a pure vertical difference.

Table 5-9: Extreme residual water levels for Mossel Bay (Source: Adapted from Theron 2016)

Return period	Residual above MSL (m)
1	0.55
25	0.89
50	0.93
100	0.97

Thus, the extreme SWL is determined by superimposing a return period storm surge and wave setup on a MHWS event and accounting for SLR. The SLR predictions up until year 2100 are discussed in Section 2.5.1. The central best estimate of SLR from various literature was determined by Theron (2016) and will be used in this study. See Table 5-10 for the calculated extreme SWL for the Mossel Bay area. The wave setup was calculated as $0.2 \cdot H_{m0}$ (Karsten Mangor 2008). The deep-water significant wave height, H_{m0}, is described in Table 5-5.

Table 5-10: Extreme still water levels expected in Mossel Bay

Return period	MHWS (m MSL)	Residual (m)	Wave setup (m)	Extreme Water levels (m MSL)			
			Year	Present	2030	2050	2100
			SLR	0 m	0.15 m	0.35 m	1 m
1	1.167	0.55	1.74	3.46	3.61	3.81	4.46
50	1.167	0.93	2.4	4.49	4.6	4.84	5.49
100	1.167	0.97	2.52	4.65	4.80	5.00	5.65

The extreme SWLs in Table 5-10 are relevant for run-up calculations for the beach at the Great Brak estuary (closed mouth condition). The wave setup values are not, however, applicable for enclosed areas, like estuaries. The effect of wave setup will then be neglected for assessment of extreme SWLs possible in the estuary under open mouth conditions. The combined effect of the inverse barometric setup and the wind effects in the residuals will still be accounted for. See Table 5-11 for the extreme SWLs with the wave setup neglected. These values will subsequently be used to determine the extreme wave loading on a structure in the lower estuary basin.

Table 5-11: Extreme still water levels expected in Mossel Bay without the effects of wave set-up

Return period	MHWS (m MSL)	Residual (m)	Future extreme Water levels (m MSL)			
		Year	Present	2030	2050	2100
		SLR	0 m	0.15 m	0.35 m	1 m
1	1.167	0.56	1.72	1.87	2.07	2.72
25	1.167	0.89	2.05	2.20	2.40	3.05
50	1.167	0.93	2.09	2.24	2.44	3.09
100	1.167	0.97	2.13	2.28	2.48	3.13

5.4 Run-up

The two models for assessing wave run-up used in this study was discussed in Section 2.7.4. The wave run-up for the shoreline in Mossel Bay was calculated for extreme events with various return periods and at different periods to account for projected SLR. A brief calculation example for a single set of parameters and each of the run-up models used can be seen in Box 5-1. The full set of results can be seen in Table 5-12.

Box 5-1: Calculation example for a single set of input parameters

Nielsen and Hanslow (1991):

Conditions:

- Year 2100 – 1 m SLR;
- 100-year extreme SWL condition, +5.65 m MSL - Table 5-10
- 100-year reverse shoaled wave, H_S = 8.28 m, T_P = 13.6 m MSL - Table 5-4

The beach slope was determined from beach profile surveys done at Great Brak in 1990 by CSIR.

The beach slope was calculated to be:

$$\tan\alpha = \frac{1}{50} = 0.02 < 0.06$$

Thus Equation 2-18 holds:

$$R_{u2\%} = SWL + 1.98 \cdot \left(0.04 \cdot \sqrt{\beta}\right)$$

Where:

$$SWL = +5.65\text{m MSL}$$

$$L_0 = \frac{gT_P{}^2}{2\pi} = 288.77\,m$$

$$H_{0rms} = \frac{H_s}{1.416} = 5.84\,m$$

$$\beta = \frac{H_{0rms}}{\sqrt{2}} \cdot L_0 = 1193.3$$

Thus,

$$R_{u2\%} = \ +8.39\,m\,MSL$$

Mather *et al* (2011):

- Year 2100 – 1 m SLR;
- 100-year extreme SWL condition, +5.65 m MSL - Table 5-10
- 100-year deep-water wave, H_{m0} = 12.6 m - Table 5-5

$$R_{u2\%} = SWL + C \cdot H_0 \cdot \left(\frac{15}{X_{15}}\right)^{2/3}$$

Where:

$$SWL = 5.65\text{m MSL}$$

$$H_0 = 12.6\ m$$

$$C = 5\ for\ large\ embayment's$$

The horizontal distance to the - 15 m contour, X_{15}, was measured to be 900 m from the SAN 123 nautical chart.

$$X_{15} = 900\ m$$

Thus,

$$R_{u2\%} = +9.76\ m\ MSL$$

Table 5-12: Calculated present and future period run-up elevation levels (m MSL) for the Great Brak shoreline

		Return period		
	Period	1	50	100
Nielsen and Hanslow (1991) (m MSL)	Present	5.38	7.10	7.39
	Year 2 030	5.53	7.25	7.54
	Year 2 050	5.73	7.45	7.74
	Year 2 100	6.38	8.10	8.39
Mather et al (2011) (m MSL)	Present	6.30	8.41	8.76
	Year 2 030	6.45	8.56	8.91
	Year 2 050	6.65	8.76	9.11
	Year 2 100	7.30	9.41	9.76

The extreme wave response 2% exceedance run-up levels, $R_{u2\%}$, can be seen in Table 5-12. The two models differ slightly as the Mather *et al* (2011) formulation estimates the run-up elevations more than 1 m higher than the Nielsen and Hanslow (1991) formulation, especially for the larger storm conditions. The elevations for all the extreme events exceed the 5 m MSL level, which is alarming as most of the Island is deemed to be under the 5 m MSL level. Overtopping of the beach berm, when closed at Great Brak is highly likely, as the berm is normally within the 2-3 m MSL range, which will fill up the estuary basin and make it possible for large waves to reach the Island, especially if a large portion of the berm is at a low elevation during an open mouth condition.

5.5 Overtopping of the berm

It is shown in Table 3-8 in Section 3.4 that the mouth condition plays a significant role in flooding events. A closed mouth during a fluvial flood can cause high water levels that would not have been reached if the mouth was open to the ocean. Similarly, the mouth condition can play an important role in extreme ocean events. Extreme tidal levels and large waves can be dissipated by the berm during a closed mouth condition. If the berm were to be overtopped by large waves, it will cause a significant increase in water level (like event 2 and 4 in Table 3-8).

An overtopping analysis will be performed to assess the viability of the Vegetated Dune concept introduced in Section 4.3.1 and to derive an estimate of a maximum volume and unit discharge of an overtopping storm condition.

5.5.1 Berm assumptions

To do an overtopping analysis of the beach berm, some assumptions must be made regarding the geometry of the beach:

- The "toe" of the beach is taken at 0 m MSL.
- The beach berm can be viewed as a smooth, impermeable slope – simplification to allow for a conservative first order estimation of overtopping rates and an overestimation is expected.
- The beach slope is taken as a constant 1:50, determined from beach profile surveys performed in 1990 (Council for Scientific and Industrial Research 1990)
- The berm crest height was chosen as the variable and the subject of analysis.

5.5.2 Overtopping unit discharge

The method for assessing overtopping unit discharges, q (m³/s/m), is discussed in Section 2.7.5.1. The calculations were done with the berm crest height (above MSL) chosen as a variable. The calculations were done for different return period storm conditions and for different periods of time, to account for the projected SLR.

Equation 2-20 and Equation 2-22 are the two relevant formulae, and will be used to calculate the overtopping discharges. See Box 5-2 for an example of the calculation for the 100-year storm condition in year 2100 when the berm is 2.5 m MSL high.

Box 5-2: Calculation example for a single set of input parameters

<u>Conditions</u>

- Year 2100, 100 – year Extreme SWL = +5.65 m MSL - Table 5-10
- 100 – year wave:
  - H_{m0} = 12.6 m, T_P = 12.5 s
- Berm crest height = 2.5 m MSL
- Beach "toe" chosen at 0 m MSL
- Beach slope, $\tan\alpha = 0.0196\ (1:50)$

The freeboard, R_C, needs to be calculated along with the iribarren number, $\xi_{m-1,0}$, to identify the relevant formula to use for the overtopping unit discharges, q (m³/s/m).

$$R_C = 2.5 - 5.65 = -3.15\ m$$

The significant wave height at the toe of the beach is calculated by interpolating between the Adjusted Goda (2000) values in Table 5-8, for the depth of 4.36 m

$$H_{mo_{toe}} = 3.23\ m$$

$$T_{m-1,0} = \cdot \frac{T_P}{1.1} = 11.36\ s$$

$$L_{m-1,0} = \frac{g T_{m-1,0}{}^2}{2\pi} = 201.44\ m$$

$$\xi_{m-1,0} = \frac{\tan\alpha}{\sqrt{H_{mo_{toe}} / L_{m-1,0}}} = 0.155$$

Thus, Equation 2-22 holds:

$$q = q_{overflow} + q_{overtopping} = 0.6 \cdot \sqrt{g \cdot |-R_C{}^3|} + 0.0537 \cdot \xi_{m-1,0} \cdot \sqrt{g \cdot H_{mo}{}^3}$$

$$q = 10.53 + 0.163 = 10.69\ m^3/s/m$$

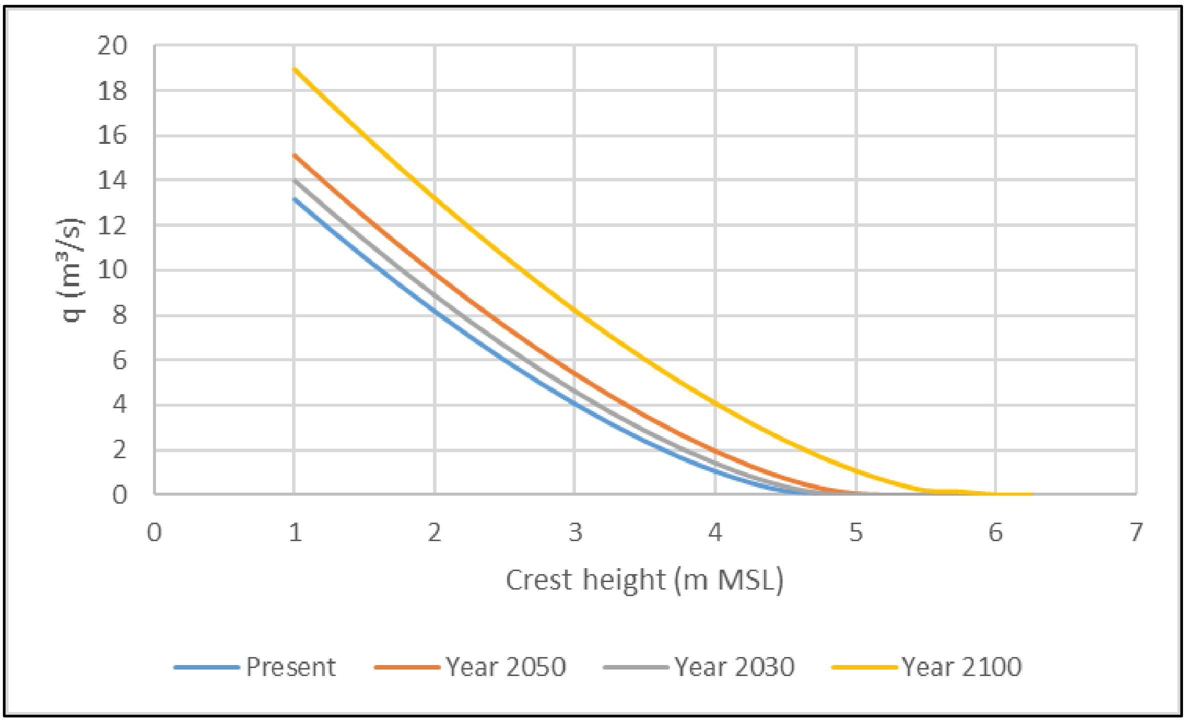


Figure 5-6: 100-year storm event overtopping discharge versus berm crest height

The unit discharges do not yet account for storm duration and probability of overtopping per wave, which will be incorporated in the maximum overtopping analysis in the following Section. See Figure 5-6 for the calculated overtopping unit discharges for the 100-year storm condition at different periods of the century.

5.5.3 Maximum overtopping volume

The method used to calculate the distribution of the maximum unit overtopping volume for a storm, V_{max}, is described in Section 2.7.5.2. The calculations were done by taking the crest height as a variable. The storm duration was taken as a constant 1 hour, as it was assumed that the extreme water level will drop after 1 hour of high tide. The unit of V_{max} then becomes *m³/m/hour*, with the hour referring to the storm duration. See Figure 5-7 for the results of the V_{max} calculations. See Box 5-3 for an example of calculation for a berm crest height of +2.5 m MSL.

Box 5-3: Calculation example for a single set of input parameters

<u>Conditions</u>

- Year 2100, 100 – year Extreme SWL = +5.65 m MSL
- 100 – year wave:
  - H_{m0} = 12.6 m, T_P = 12.5 s
- Berm crest height = 2.5 m MSL
- Beach "toe" chosen at 0 m MSL
- Beach slope, $\tan\alpha = 0.0196 = \frac{1}{50}$
- Storm duration = 1 hour

Equation 2-23 to Equation 2-26 are the relevant formulae for the calculation of the probable maximum overtopping unit volume per hour, V_{max}. The probability of overtopping per wave, P_{ov}, can be calculated as:

$$P_{ov} = exp\left[-\left(\sqrt{-\ln 0.02}\,\frac{R_C}{R_{u2\%}}\right)\right] = 0.147$$

With $R_C = -3.15\ m$, from Section 5.5.2 and $R_{u2\%}$ calculated for smooth, impermeable slopes from the CEM (2006) as:

$$R_{u2\%} = H_{S_{toe}}(A\xi_{m-1,0} + C)\gamma_r\gamma_b\gamma_b\gamma_\beta = 5.17\ m$$

With A = 1.6 and C = 0, $\xi_{m-1,0} = 0.155$ from Section 5.5.2 and all the reduction factors, γ, assumed to be 1.0. The number of waves expected to overtop the berm can then be calculated as the product of the probability of overtopping and the total number of waves. The total number of waves are calculated using the storm duration and the mean period of the incident wave, $T_{m-1,0}$= 11.36 s (Section 5.5.2). In one hour, a total number of 317 waves are expected.

$$N_{ov} = P_{ov} \cdot N_w = 45$$

The maximum overtopping volume per meter for 1 hour can then be calculated using Equation 2-23:

$$V_{max} = a \cdot [\ln(N_{ov})]^{\frac{4}{3}}$$

Where a is calculated as follows using q = 6.88 m³/s/m:

$$a = 0.84 \cdot T_m \cdot \frac{q}{P_{ov}} = 892.85$$

Then the maximum volume to overtop the berm for the 100-year event, during the hour-long spring high tide can be calculated as:

$$V_{max} = 5\ 307\ m^3/m$$

The results for V_{max} shows that the berm crest height plays a vital role in protecting the estuary from flooding caused by overtopping waves. The length of the berm was measured to be approximately 170 m (Google Earth 2017). Which makes the maximum overtopping volume of the 1 hour, 100-year storm event approximately 902 190 m³. The overtopping volume is not as large as a fluvial flood can be. The volume of the 1993 flood was 11.3 Mm³ and 44 Mm³ for the 2007 event (Roux, Rademeyer 2012). However, the proximity of the Island may still render it vulnerable to possible wave attack if the mouth is open and to flooding caused by the raised estuary water level.

Unfortunately, there is limited information regarding events of berm overtopping which makes validation of the results very hard. The results, however, are deemed to be an adequate estimation and will be used in this study to further investigate the validity of potential flood defence measures and not to derive design conditions.

The effect of shortening the berm length that can be overtopped can be assessed using the results presented in Figure 5-7.

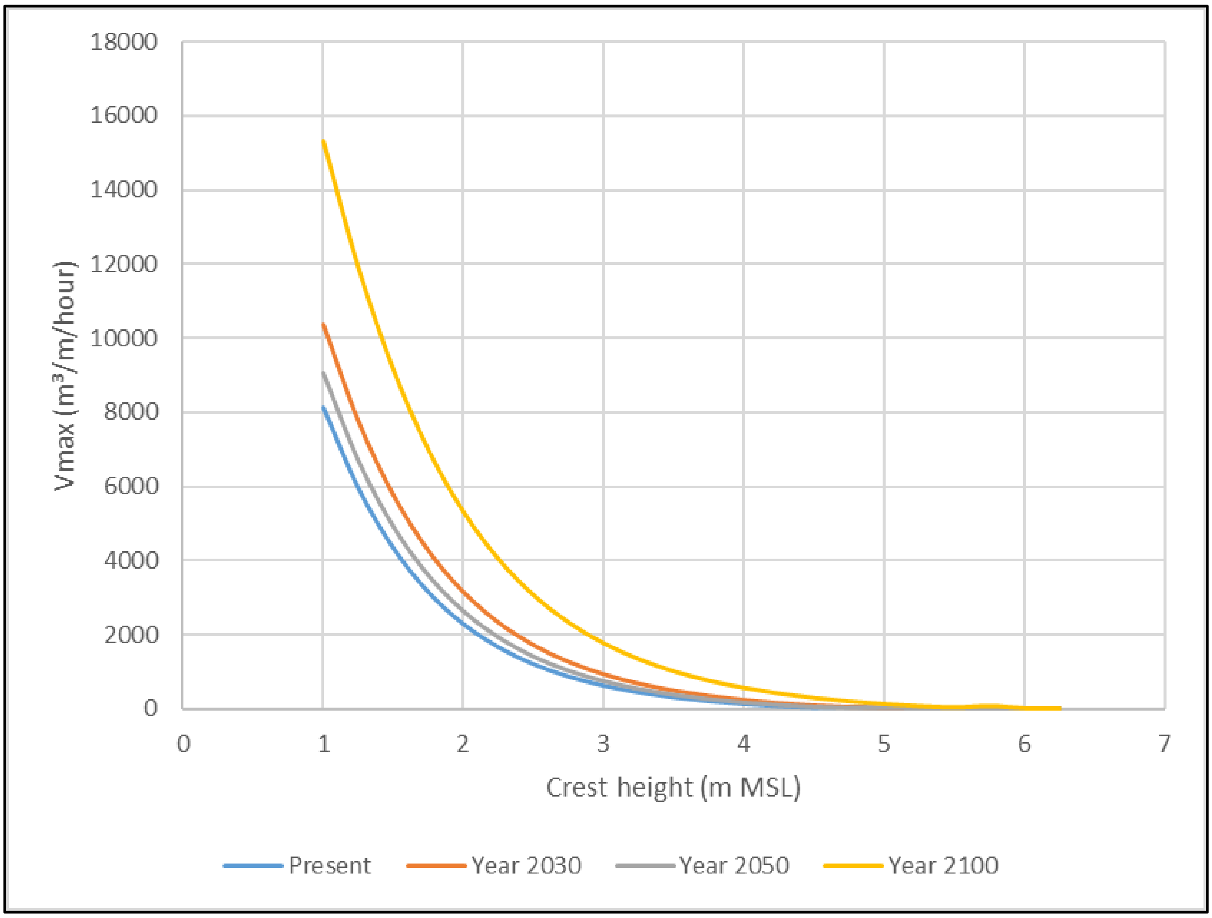


Figure 5-7: 100-year storm event maximum overtopping volume versus berm crest height

5.5.4 Effect of shortening the overtopping length

A vegetated dune concept was presented in Section 4.3.1 as a method to limit the overtopping volume over the mouth berm during large storm events. The concept is comprised of using methods to stabilise a dune to achieve a certain crest level. The length of the dune cannot be so long as to interfere with a desired width of inlet channel, thus a vegetated dune cannot be made to cover the entire berm length (See Figure 5-8). Figure 5-7 was used to calculate the effect of a shorter berm length that can be overtopped on the maximum overtopping volume during a 100-year storm in year 2100, with an assumed duration of 1 hour (See Table 5-13). The crest height of the length of berm that can be overtopped (without the vegetated dune) is chosen as 2 m MSL.

Table 5-13: Effect of a shorter overtopping length on 100-year storm conditions in year 2100 and a +2 m MSL berm crest height

Overtopping length (m)	Length of dune – L_D (m)	V_{max} (m^3)
170	0	902 243
150	20	796 097
130	40	689 950
110	60	583 804
90	80	477 658
70	100	371 512
50	120	265 366

Figure 5-8: Vegetated dune concept (Google Earth 2017)

It is very interesting to note, that for this specific case, the maximum overtopping volume for the hour storm is approximately halved from the base situation (no dune) due to a mere 80 m of dune length. These results will be taken as proof of concept, that a vegetated dune with an adequate crest height will help to protect the lower estuary basin from flooding driven by large wave events and storm surge. A smaller possible inlet channel will further limit wave penetration into the estuary and protect the Island by wave-attack driven inundation.

5.6 Assessment of possible extreme water levels in the lower estuary basin

The possible extreme water levels due to marine and fluvial related flooding will be discussed in this section to be used to identify flood vulnerable areas and to establish approximate design water levels and wave conditions.

5.6.1 Marine flooding

An extreme water level is possible in the lower estuary basin due to marine hydrodynamics and can be achieved by a tidal intrusion of raised still water levels during an open mouth condition and by overtopping of the estuary mouth berm when the mouth is closed. The driver of coastal flooding was found to be mainly large ocean storm events. Climate change predictions from the literature was incorporated to assess possible extreme Still Water Levels for various periods of the century. The joint probability of occurrence for extreme SWL components were assessed in a rudimentary method of superimposing likely events, due to the absence of long period water level recordings.

The MHWS event was chosen as the primary tidal component, as the joint probability of occurrence of a large ocean storm during a MHWS event was deemed likely enough for consideration as the conservative design condition. The extreme Still Water Level for the present and future periods can be seen in Table 5-10 and Table 5-11. The wave setup component was neglected from the extreme SWL calculated in Table 5-11, as it is less applicable for sheltered areas, i.e. estuaries.

During closed mouth conditions, large waves overtopping the estuary mouth berm can potentially raise the water level in the lower estuary basin to the level of the berm, where it will start to breach. The highest recorded berm height was in the order of +3.8 to +4.5 m MSL after a long period of mouth closure due to severe drought conditions, in 2010. From the overtopping analysis in Section 5.5 it is apparent that the 100-year storm wave, in the year 2100, will overtop the estuary mouth berm if it is under +6.25 m MSL. However, a very long storm duration will be needed to cause the volume of water that overtop the berm to cause extensive flooding, as the relatively high crest height will offer a degree of protection.

Thus, the tidal intrusion during an extreme storm event and open mouth will be taken as the governing, worst-case scenario event for flood water levels in the estuary due to marine hydrodynamics and for wave penetration. Assumptions regarding the maximum depth of water in the estuary will govern the size of the depth-limited significant wave height possible at the Island. The extreme SWL calculated in Table 5-11 will be taken as the achievable water levels in the estuary due to tidal intrusion, and coupled with a conservative assumption of a minimum inlet bed level, the consequent water depth can be used to calculate the design wave heights that may reach the Island. This assumption is discussed later in the section.

5.6.2 Fluvial flooding

5.6.2.1 Past flood events and expected future floods

Past flooding events have been described in Section 3.5. It has been shown that the mouth state, berm height and initial dam- and estuary water level prior to a large rainfall event has a significant effect on the achievable extreme water level in the estuary. A 20-year storm peak of 404 m^3/s, the largest experienced at Great Brak, coincided with an open mouth and only raised the water level to +2.2 m MSL, whereas a 10-year storm peak of 336 m^3/s in 2011, which coincided with a closed mouth, caused a water level in the lower estuary basin of +2.9 m MSL and property damage to low-lying development around and in the estuary.

The catchment is deemed to experience even larger flooding events as shown in Section 5.1.1. The 100-year design flood peak for the event of a 100% full dam was calculated as 865 m^3/s. Therefore, even higher water levels will be achievable in the lower estuary basin, especially if the mouth is closed prior to the flood or when the fluvial flood coincides with an open mouth and elevated ocean still water levels. The effect that the 100-year flood event has on water levels in the lower estuary basin during a closed mouth condition as well as the effect of the river run-off – high ocean level interaction during open mouth conditions needs to be assessed to obtain the design water levels for a viable flood defence at the Island.

The Emergency Protocol of the estuary is important in the event of an expected river flood. During the worst-case scenario of a full dam prior to the flood, there will be water available to breach the estuary mouth, which will allow the flood to pass into the ocean. Currently, if the dam level is at 70%, chances are that there will be no water release to open the mouth, as the Water Release Policy only allows an annual release of 1 million m^3 when the dam is below 70%. The river flood will experience some attenuation from the dam and will breach the berm at +2.0 m MSL, as per the Emergency Protocol and Estuary Management Plan. The level of the sea at the moment of breaching will either help or hinder the outflow of water into the ocean. A too high downstream water level will cause additional damming in the estuary. The 100-year extreme ocean SWL in 2100, calculated to be possible in sheltered areas like an estuary, is equal to +3.13 m MSL, which already will cause flooding. If this extreme ocean SWL were to coincide with river flooding, water levels may rise to over +4.0 m MSL.

From historic events and assessment of the Emergency Protocol some important conclusions can be made regarding possible water levels. A water level of over +3.5 m MSL is deemed to be possible during adverse conditions. A combination of adverse situations, like a full dam and a closed mouth in a large rain storm, will lead to unforeseen flooding levels given the close proximity of the dam to the town of Great Brak estuary. To account for this uncertainty, various design crest heights for fluvial defence will be analysed, namely: +3.0 m MSL, +3.5 m MSL, +4.0 m MSL and +4.5 m MSL.

5.6.2.2 Estuary basin model and hydrodynamic modelling of the estuary

The Authorities have been using a water level – volume relationship, calculated by Huizinga (n.d.) from bathymetric survey data to calculate the required volume of water to be released from the dam to raise the water in the lower estuary basin to a desired water level, mainly for the purpose of breaching the mouth berm. This water level – volume relationship was obtained from Huizinga (2017) and was subsequently used in the setup of a one-dimensional estuary basin model, using the fundamental principles of conservation of mass and momentum as discussed in Sections 2.3.7 and 2.3.8. This estuary basin model approach was used to calculate a first order approximation of possible extreme water levels in the estuary under fluvial flooding conditions

This method proved to be incapable of describing the complex scouring process of the estuary mouth under large discharges, and therefore yielded incorrect water levels. Water levels of between +12 m MSL and +38 m MSL were obtained under various realistic inlet geometry assumptions. This approach was discarded due to the incapability of describing the complex relationship between the inlet geometry and the discharge through the mouth, as the flow through the estuary mouth can be classified as unsteady and non-uniform.

To accurately assess the extreme water levels in the lower estuary basin, a river hydraulic and morphological numerical modelling software package (e.g. MIKE 21C by DHI) could potentially be used to account for the scouring of the mouth area. Due to the wider objectives of the study, the hydraulic and morphological modelling of the estuary reach was deemed to be outside of the scope of work. It will, however, be considered as a recommendation – to establish the various return period flood lines and subsequently for adequate Setback line delineation.

The Great Brak estuary has in the past been modelled using one-dimensional hydrodynamic modelling techniques (see Section 3.10) which incorporated fluvial and marine flooding components. These studies were performed by inexperienced persons who made a few simplifying assumptions to account for the estuary mouth geometry and the meandering river upstream of the lower estuary basin, thus the reliability of the results as design values is uncertain. However, in the absence of detailed hydrodynamic and morphological modelling of the estuary to establish the flood lines, these studies will be used as a first order estimate of achievable extreme water levels and together with the conclusions drawn from the previous section to create vulnerability criteria regarding elevations for the perimeter of the Island. A summary of the studies' results for the case of a 100 – year storm (combined fluvial and marine flood components) plus a sea level rise of 1 m during open mouth and closed mouth conditions can be seen in Table 5-14.

Table 5-14: Summary of calculated extreme water levels for relevant design conditions in the Great Brak estuary

Pieterse (2014)	**Closed mouth condition + breaching at +2 m MSL**
	Lower Estuary basin = +3.3 m MSL
	Open mouth condition
	Lower Estuary basin = +3.6 m MSL
Du Pisani (2015)	**Closed mouth conditions**
	Lower Estuary Basin:
	Barrier @ -1 m MSL+MHWS = +3.1 m MSL
	Barrier @ +1 m MSL = +3.2 m MSL
	Barrier @ +2 m MSL = +3.4 m MSL
	Barrier @ +3 m MSL = +3.9 m MSL
	Barrier @ +4 m MSL = +4.6 m MSL
	Open mouth conditions
	Barrier @ -1 m MSL+MHWS+SS+SLR = +3.3 m MSL
	Barrier @ 0 m MSL+MHWS+SS+SLR = +3.4 m MSL

From considering the abovementioned results and the conclusion regarding tidal intrusion, from section 5.6.2.1, the following vulnerability to flooding criteria for ground elevation of development on the Island was derived for the design conditions. The vulnerability to flooding due to low elevation will be assessed using the following criteria set out in Table 5-15. The vulnerability criteria will be used in hazard mapping for the Island in subsequent sections. Areas with an elevation of less than 2.5 m MSL is deemed to have a very high vulnerability to flooding, where areas with elevation over +5 m MSL are deemed to have a very low vulnerability to flooding.

Table 5-15: Vulnerability criteria for flooding regarding the elevation (m MSL) of development on the Island.

Vulnerability	Very low	Low	Medium	High	Very high
Ground elevation of area assessed	>5	> 4.5 ≤ 5	> 3.5 ≤ 4.5	> 2.5 ≤ 3.5	> 1 ≤ 2.5

5.7 Flood hazard assessment

Coastal hazard assessment is a tool for quantification of vulnerability based on an expert analysis of functional responses. This tool was used by Theron *et al* (2012) to assess the vulnerability to coastal hazards of the coast of Mozambique. Theron *et al* (2012) used a methodology adapted from Coelho *et al* (2006), describing it as "pragmatic" and most relevant to the southern African context. An adaptation of the Coelho *et al* (2006) method will be used to assess the vulnerability of the Island to various flooding scenarios. This method will help identify the areas of the Island that are vulnerable to flooding which will need defending.

5.7.1 Adaption of hazard assessment method

The method prescribed by Coelho *et al* (2006) identifies nine indicators of coastal vulnerability which are: foreshore elevation, distance of development to shore, tidal range, maximum wave height, erosion/accretion rate, geology, geomorphology, ground cover and anthropogenic actions. Some of these indicators are irrelevant to the flood hazard assessment for the Island, as the Coelho method is focussed on hazard assessment for large stretches of coastline with many variables. The method for assessing the flooding vulnerability of the Island in context will need an adaptation to the methods described in Coelho *et al* (2006) and in Theron *et al* (2012).

From assessing the potential extreme flooding conditions at the Island in preceding sections, the following potential hazard indicators and scenarios were identified and will be used in the vulnerability mapping for the Island. The method to be used for hazard mapping entails the assessment of each point (location) against the vulnerability parameters below for various scenarios. These scenarios are chosen and implemented by means of adjusting the weighting of specific vulnerability parameters. The elevation of the development on the Island, the vulnerability to direct wave attack through the estuary mouth, local ground cover and geology was chosen as the relevant flood vulnerability parameters to be assessed. The vulnerability to wave attack will be assessed as a rating between 1 (very low) and 5 (very high). Only the south-western perimeter of the Island can be reached by waves penetrating the estuary mouth. See Table 5-16 for the vulnerability assessment criteria.

Table 5-16: Vulnerability criteria for flood hazard mapping

Vulnerability criteria	Very low	Low	Medium	High	Very high
	1	2	3	4	5
E – elevation above mean sea level	> 5	> 4.5 ≤ 5	> 3.5 ≤ 4.5	> 2.5 ≤ 3.5	> 1 ≤ 2.5
WH – vulnerable to direct wave attack	1	2	3	4	5

GC – ground cover	Forest	Ground vegetation	Non – covered	Rural urbanized	Urbanized
GL– geology	Magmatic rocks	Metamorphic rocks	Sedimentary rocks	Non-consolidated coarse sediments	Non – consolidated fine sediments

Two main drivers of flooding were identified as fluvial flooding and coastal storm induced flooding (tidal intrusion and wave attack). The mouth state, during these extreme events was found to be a significant factor and will be accounted for in the scenarios. The scenarios are implemented by means of a weighting matrix. The various weights act to cancel out or reduce/increase the significance of the specific vulnerability parameter during each scenario. Ten different scenarios were identified which accounts for fluvial and coastal flooding during open and closed mouth conditions. The coastal flooding will be assessed for different periods, to consider the rising sea levels. The 100-year return period flood, for both fluvial and coastal events, is chosen as the hazard to be assessed in each scenario. See Table 5-17 for the various scenarios to be considered.

Table 5-17: Various scenarios to be assessed during the hazard assessment

Scenario	Open mouth condition - 11	Closed mouth condition - 21
Fluvial flooding - A	A1	`A2
Coastal flooding - B	Present: B11 Year 2030: B12 Year 2050: B13 Year 2100: B14	Present: B21 Year 2030: B22 Year 2050: B23 Year 2100: B24

The various scenarios will render certain vulnerability parameters less or more relevant. For example, when the closed mouth condition and fluvial flood scenario is considered, the wave attack parameter is chosen as zero, and the elevation parameter enjoys most of the weighting. Similarly, for the scenario of coastal flooding and open mouth conditions, the wave attack parameter becomes more significant than the elevation. The elevation and wave attack vulnerability weighting increases with the scenarios envisioned for later in the century, due to the rising sea levels and projections for an increase in storminess. The elevation parameter carries more weight during all closed mouth conditions to account for damming and during open mouth conditions the vulnerability against wave attack governs. The geology and ground cover are the least weighted parameters in all cases. See Table 5-18 for the scenario weighting matrix used.

Table 5-18: Scenario vulnerability parameter weighting matrix

Scenario	**E**	**WH**	**GC**	**GL**
A1	3	2	2.5	2.5
A2	5	0	2.5	2.5
B11	2	4	2	2
B12	2.3	3.7	2	2
B13	2.6	3.4	2	2
B14	3	4	1.5	1.5
B21	4	3	1.5	1.5
B22	4.3	2.7	1.5	1.5
B23	4.6	2.4	1.5	1.5
B24	5	2	1.5	1.5

5.7.2 Vulnerability mapping

The vulnerability mapping was done by dividing the perimeter of the Island into 91 sections. The elevations corresponding to each section were obtained from the 2005 detailed bathymetry survey (See Figure 3-8). The ground cover was found to be either "Rural urbanised" or "Urbanised" and the geology was identified as "Sedimentary rock" for the areas where there is development (i.e. to account for the consolidation) and as "Non-consolidated fine sediment" where there is no development. The most vulnerable areas to flooding can subsequently be identified from combining the overall vulnerability map (including all the scenarios and parameters), the wave attack vulnerability map and the elevation vulnerability map. The colour vulnerability scale used can be seen in . The elevation vulnerability map can be seen Figure 5-10, the wave attack vulnerability map in Figure 5-12 and the overall vulnerability map Figure 5-11.

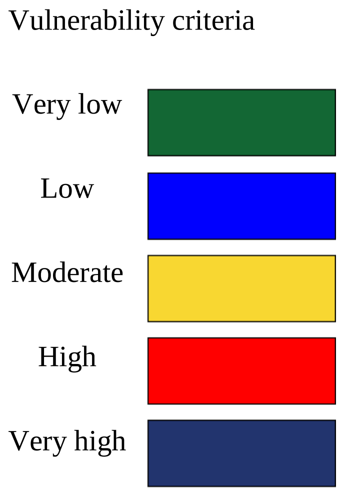


Figure 5-9: Colour vulnerability scale

From the vulnerability maps generated for the Island, it can be seen that there are very few areas around the Island that are not vulnerable to flooding from some scenario. The elevation vulnerability map (Figure 5-10) shows that most of the Island perimeter is vulnerable to flooding from extreme water levels in the estuary (Red and Purple), where only the southern part of the Island is elevated enough to receive a Low and Moderate vulnerability rating. The southern part of the Island, however, is vulnerable to direct wave attack during open mouth conditions (Figure 5-12).

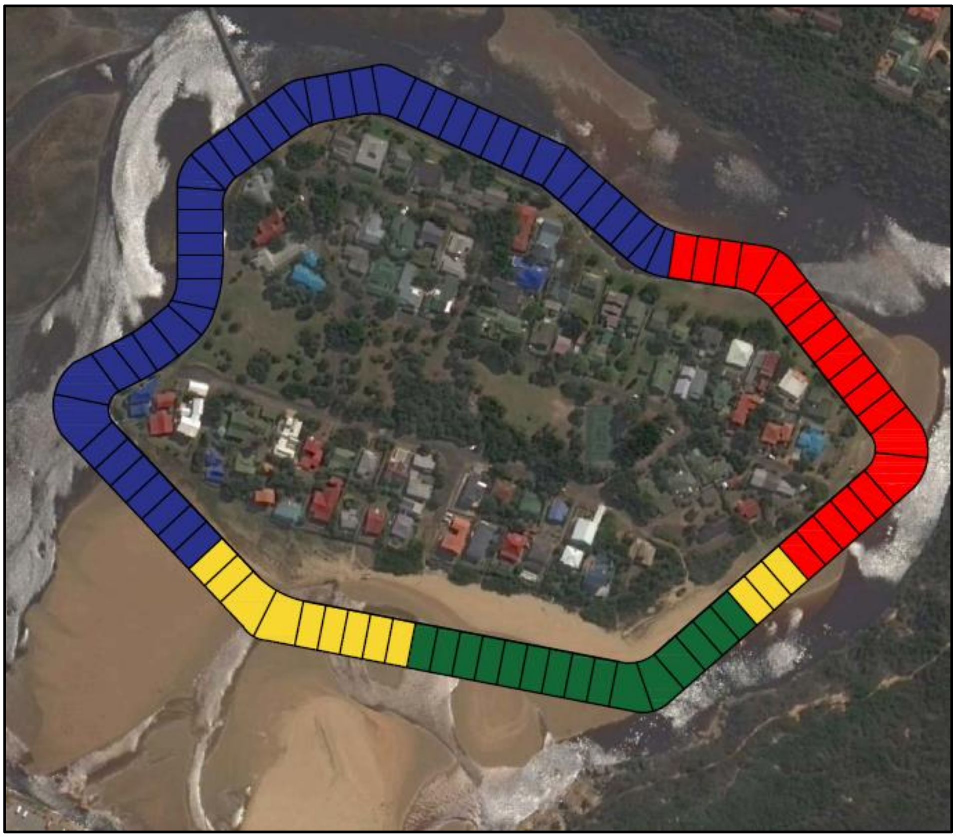


Figure 5-10: Elevation vulnerability mapping

The overall vulnerability map (Figure 5-11), where various scenarios are regarded, shows that the majority of the Island is moderately vulnerable to flooding. There is only a small portion of the Island that received a Low vulnerability rating. Therefore, the flood defence measure would need to alleviate flood conditions for the whole perimeter of the Island to be deemed successful.

Figure 5-12: Wave attack vulnerability mapping

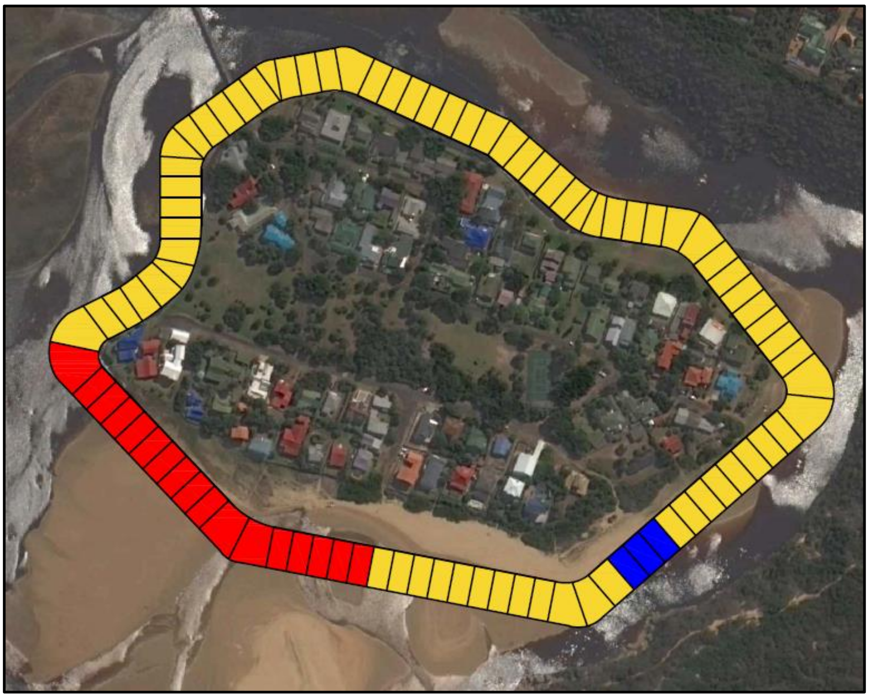


Figure 5-11: Overall vulnerability mapping

5.8 Conclusion

5.8.1 Fluvial flooding

The possible extreme fluvial flood peaks for the catchment K20A was assessed using standard hydrological methods and by considering the attenuation effect of the Wolwedans Dam. Under the current water release policy, there is no provision made for flood attenuation, thus the 100- and 50-year return period flood peaks were calculated with the assumption of the Wolwedans Dam at 100%. The extreme flood peaks for the whole catchment (i.e. what the lower estuary basin can experience if the whole catchment contributes to flooding) were calculated as 865 m^3/s and 682 m^3/s respectively.

A one-dimensional basin model for the Wolwedans Dam was also developed to assess the degree of attenuation for various dam levels. From Figure 5-3 the relationship between the initial dam level (before commencement of the flood) and the degree of attenuation can be seen. It should be noted that the attenuation percentage is calculated using the inflow and outflow flood peaks, and not storm water volume. The volume of storm water is dependent on the storm duration where the flood peak will be achieved following a storm duration that is an increment of the catchment time of concentration. Roux and Rademeyer (2012) suggested the storm duration of $2*t_C$ should be used to assess the extreme flood peaks for the catchment. It is however possible for a storm with a longer duration to deliver a much larger volume of water and be attenuated to a lesser degree, but the flood peak of such a flood will not be larger than for the $2*t_C$ duration event. Figure 5-3 can therefore not be used as an accurate indication of flood volume attenuation, but rather to conceptualise the influence of a less than full dam on passing extreme flood peaks.

The influence of the dam (or a less than full dam) was found to be significant enough to merit investigation into the application of flood attenuation measures at the upstream hydraulic control, which will benefit the whole estuary reach. Table 5-2 shows the volume of water to release to increase the attenuation effect of the dam. Very large releases must be made from a full dam to significantly increase the attenuation effect. For example, if a 20 % attenuation effect is required (an additional 10 % from a full dam condition), a 3.7 million m^3 (50-year flood) and 5 million m^3 (100-year flood) of water must be released from a full dam prior to the flood. The added attenuation will reduce the respective extreme floods to 20- and 50 – year floods. The amount of water to be released to achieve a 10 % increase in attenuation effect is deemed to be too much for the benefits it provides. It has also been shown that ensuring an open mouth prior to a fluvial flood, by means of a water release from the dam, can seriously limit the achievable water level in the estuary (refer to the flood in November 2006), which will require significantly less water to be released from the dam (than flood attenuation). Thus, lowering the dam level as a direct flood defence measure is deemed not to be a viable option and will not be evaluated further.

5.8.2 Coastal flooding

During open mouth conditions, if a deep and wide enough inlet channel is formed or a large enough connection with the ocean, such as during an extreme Still Water Level event, it is deemed possible for waves to penetrate the estuary mouth and reach the Island. The calculated extreme run-up levels show that low lying properties close to the shore may experience flood damage if below the +5.3 m MSL contour line under present conditions and below the +8.3 m MSL contour line in the year 2100 for the 100-year storm.

The depth-limited significant wave height calculated in Section 5.2.2.3 for various extreme waves is the estimation of wave heights in depth-limited conditions and can be used to assess the extreme wave height at the Island under adverse conditions. Adverse conditions will exist if a normal (i.e. perpendicular) incident wave angle and an open mouth condition that is deep and wide enough to allow wave penetration. The water depth will be subject to the inlet mouth depth; thus, the width of the estuary mouth will be assumed wide enough to allow wave penetration, as a conservative assumption. It is likely that, due to the relatively narrow estuary mouth entrance and the presence of a shallow sand bar, the wave energy will be dissipated more than initially expected due to further breaking, diffraction and refraction.

The lowest water level in the lower estuary basin after a flood event was recorded as -0.277 m MSL after the November 2007 storm. The mouth was breached by a flush release, two days prior to the storm which allowed for the flood to flow into the ocean without causing flooding to low-lying properties. The storm peak that spilled over the dam correlated with a 20-year storm after being attenuated from a 100-year storm by the fortunately only 65% full dam. Much larger storms are predicted to hit the catchment area in the next century and if the dam is full and the normal emergency protocol of opening the mouth is followed, it may be possible for the mouth to be flushed open deeper than -0.277 m MSL. A conservative design inlet channel bed elevation will be assumed to be -0.5 m MSL. Thus, the water depth in the estuary mouth will be assumed to be from -0.5 m MSL to the extreme SWL calculated in Table 5-11. By interpolation of the extreme depth-limited significant wave height results, in Table 5-8 for the Adjusted Goda (2000) formulation, a significant wave height, H_S, can be estimated. This wave height will be taken as the design wave height for the design of the flood protection measure.

Overtopping of the berm height during closed mouth conditions was deemed highly likely and has been observed in the past. There is no existing long record of berm height measurements, thus the berm height was assumed a variable, with a constant slope. From the berm overtopping analysis undertaken in Section 2.7.5, it is estimated that in the year 2100, a +6.25 m MSL berm crest level will stop the design storm from overtopping. From the overtopping analysis, it can be concluded that a protective fore dune can limit overtopping of the mouth berm and help protect the Island from direct wave attack. The option of dune stabilisation will be evaluated further.

6 Preferred Defence Measures

Various potential flood defence measures were identified in Section 4 and will be subject to evaluation in this section. The various potential options will be subjected to an initial evaluation in the form of an advantage/disadvantage analysis to narrow down on promising options. The three most promising options will then be subjected to an in depth Multi-Criteria Analysis (MCA). The results of the MCA will be used to identify the preferred option, which will then be designed on a conceptual basis and a final order of magnitude cost estimate will be derived for that option.

6.1 Evaluation of potential flood defence measures

The first order evaluation of the potential defence options will be in the form of an advantage/disadvantage analysis. The advantages and disadvantages of each potential flood defence measure (FDM) listed in Table 4-7 and Table 4-8, will be discussed in terms of the evaluation criteria developed for the hydraulic performance, environmental – and economic impacts. The advantages and disadvantages are identified by consulting various literature relating to shore and flood protection projects (CIRIA 2007, Heron, 2013, Theron, *et al*, 2012, USACE 2006). The advantage/disadvantage analysis will also function as a manner to contextualise the potential options in terms of the evaluation criteria.

See Table 6-1 and Table 6-2 for the evaluation of the armoured and unarmoured dike structures.

Table 6-1: Advantages and disadvantages of the armoured dike flood defence option

Description	Advantages	Disadvantages
Dike - armoured	**Hydraulic performance** ○ Effective means of defence of against fluvial flooding ○ Can be integrated into other coastal defence options ○ Can dissipate direct wave energy effectively ○ Low maintenance ○ Long lifetime **Environmental impact** ○ Can increase flushing efficiency of estuary by allowing breaching at higher water levels ○ Limited construction noise if pre-casted concrete slabs is used **Economic impact** ○ Considered to be a cost-effective solution	**Hydraulic performance** ○ Easily overtopped by waves – mitigate by adding extra crest height **Environmental impact** ○ Will influence the ebb and flood channels which can lead to habitat destruction ○ Visually obtrusive for both visitors to the coastal public property and for residents of the Island – reduce crest height by combining with other FDM ○ Not beneficial for the whole estuary reach – mitigate by systematically applying FDM to other vulnerable areas **Economic impact** ○ Larger capital cost (than unarmoured) due to armouring

Table 6-2: Advantages and disadvantages of the unarmoured dike flood defence option

Description	Advantages	Disadvantages
Dike - unarmoured	**Hydraulic performance** ○ Effective means of defence of against fluvial flooding ○ Can be integrated into other coastal defence options ○ Can dissipate direct wave energy effectively **Environmental impact** ○ Can increase flushing efficiency of estuary by allowing breaching at higher water levels ○ Can promote vegetation growth ○ Limited construction noise **Economic impact** ○ Considered to be a cost-effective solution	**Hydraulic performance** ○ Easily overtopped structure – Mitigate by adding extra crest height ○ High maintenance due to erosion from both wave attack and fluvial flooding **Environmental impact** ○ May influence the ebb and flood channels which can lead to habitat destruction and mouth meandering ○ Visually obtrusive for both visitors to the coastal public property and for residents of the Island - reduce crest height by combining with other FDM **Economic impact** ○ Not beneficial for the whole estuary reach - mitigate by systematically applying FDM to other vulnerable areas

See Table 6-3 and Table 6-4 for the evaluation of the impervious and porous revetment type structures.

Table 6-3: Advantages and disadvantages of the non-porous revetment flood defence measure

Description	Advantages	Disadvantages
Revetment – impervious (concrete)	**Hydraulic performance** ○ Considered robust coastal engineering solution ○ Effective means of defence of against fluvial flooding ○ Can be integrated into other coastal defence options ○ Can dissipate direct wave energy effectively – little to no reflection ○ Access can be granted via steps ○ Half the width of dike **Environmental impact** ○ Can increase flushing efficiency of estuary by allowing breaching at higher water levels	**Hydraulic performance** ○ Easily overtopped structure – Mitigate by adding extra crest height and roughness elements ○ Slope is still too gradual – will be take up too much horizontal space **Environmental impact** ○ May influence the ebb and flood channels which can lead to habitat destruction and mouth meandering ○ Construction method will require the placement of land fill in the estuary and dredging ○ Visually obtrusive for both visitors to the coastal public property and for residents of the Island - mitigate by reducing crest height in combination with other FDM ○ Hard to incorporate into natural estuarine setting ○ Not beneficial for the whole estuary reach - mitigate by systematically applying FDM to other vulnerable areas ○ Construction method will require access over the beach **Economic impact** ○ Typically, a very expensive defence option due to the volume of concrete

Table 6-4: Advantages and disadvantages of the porous revetment flood defence option

Description	Advantages	Disadvantages
Revetment – porous (rubble mound, gabions)	**Hydraulic performance** ○ Dynamic structure – stays functional if setting occurs ○ Very good option for wave dissipation – little to no reflection, less toe scour (rubble mound) ○ Effective means of defence of against fluvial flooding ○ Provides drainage of water from behind structure ○ Steeper slopes can be achieved with gabions – less horizontal space requirement **Environmental impact** ○ Can increase flushing efficiency of estuary by allowing breaching at higher water levels ○ Can be integrated into a natural environment (rubble mound) **Economic impact** ○ Typically cost effective– dependant on transportation cost for rock materials ○ Little to no maintenance (not gabions)	**Hydraulic performance** ○ Porous – seepage will occur when water level is too high (i.e. above the ground level of the Island) – mitigate by incorporating a non-porous back layer/retaining wall ○ Access over armour rock is limited – mitigate by supplying wooden walk ways ○ Gabion casings prone to corrosion due to e.g. rock movement **Environmental impact** ○ Construction method will require the placement of land fill in the estuary and dredging ○ Construction method will require access over the beach ○ May influence the ebb and flood channels which can lead to habitat destruction and mouth meandering ○ Gabion casings are hard to incorporate into the natural environment of the estuary – mitigate by promoting plant growth ○ Visually obtrusive for both visitors to the coastal public property and for residents of the Island - reduce crest height by combining with other FDM ○ Not beneficial for the whole estuary reach - mitigate by systematically applying FDM to other vulnerable areas **Economic impact** ○ Gabions need recurring maintenance which increases the lifetime cost

The L-shaped-, Mass Gravity- and the Gabion sea wall are evaluated in Table 6-5 to Table 6-7.

Table 6-5: Advantages and disadvantages of the L-shape concrete seawall flood defence option

Description	Advantages	Disadvantages
Sea wall – L – shaped concrete	**Hydraulic performance** ○ Considered a good coastal engineering option ○ Units can be pre-casted – faster construction ○ High grade finish can be specified due to pre-casting ○ Services can be easily accommodated in lee of structure ○ Estuary channel width can be maintained **Environmental impact**	**Hydraulic performance** ○ Not very effective for wave energy dissipation – reflection may cause local scouring ○ Steel reinforcing can corrode. Mitigate with correct design and quality control ○ Sensitive to splash up – mitigate by incorporating a recurve wall **Environmental impact** ○ Construction method will require the placement of land fill in the estuary and dredging

Description	Advantages	Disadvantages
	○ Can increase flushing efficiency of estuary by allowing breaching at higher water levels ○ Can be built to suit landscaping and beach form **Economic impact** ○ Typically, a very cost-effective solution ○ Low maintenance cost	○ Construction method will require access over the beach ○ May influence the ebb and flood channels which can lead to habitat destruction and mouth meandering ○ Visually obtrusive for both visitors to the coastal public property and for residents of the Island - reduce crest height by combining with other FDM ○ Not beneficial for the whole estuary reach - mitigate by systematically applying FDM to other vulnerable areas

Table 6-6: Advantages and disadvantages of the mass gravity concrete seawall flood defence option

Description	Advantages	Disadvantages
Sea wall – Mass gravity concrete	**Hydraulic performance** ○ Considered a robust coastal engineering option ○ Estuary channel width can be maintained ○ Services can be easily accommodated in lee of structure **Environmental impact** ○ Can increase flushing efficiency of estuary by allowing breaching at higher water levels ○ Can be built to suit landscape and beach form – thus made aesthetically pleasing ○ Access to water can be easily achieved **Economic impact** ○ Low maintenance cost	**Hydraulic performance** ○ Not very effective for wave energy dissipation – reflection may cause local scouring ○ Sensitive to splash-up – mitigate by incorporating a recurve wall ○ High concrete volume **Environmental impact** ○ Construction method will require the placement of landfill in the estuary and dredging ○ Construction method will require access over the beach ○ May influence the ebb and flood channels which can lead to habitat destruction and mouth meandering ○ Visually obtrusive for both visitors to the coastal public property and for residents of the Island - reduce crest height by combining with other FDM ○ Not beneficial for the whole estuary reach - mitigate by systematically applying FDM to other vulnerable areas **Economic impact** ○ Typically, a very expensive solution – due to the volume of concrete

Table 6-7: Advantages and disadvantages of the gabion seawall flood defence option

Description	Advantages	Disadvantages
Sea wall – Gabions	**Hydraulic performance** ○ Estuary channel width can be maintained **Environmental impact** ○ Can increase flushing efficiency of estuary by allowing breaching at higher water levels **Economic impact**	**Hydraulic performance** ○ Not very effective in wave energy dissipation. Mitigate by incorporating a curved wall structure to limit splash up ○ Sensitive to toe erosion. Mitigate by providing adequate scour cover **Environmental impact**

Description	Advantages	Disadvantages
	○ Considered good short-term solution ○ Cost-effective	○ Construction method will require access over the beach ○ Construction method will require the placement of land-fill in the estuary and dredging ○ May influence the ebb and flood channels which can lead to habitat destruction and mouth meandering ○ Not beneficial for the whole estuary reach - mitigate by systematically applying FDM to other vulnerable areas ○ Visually obtrusive for both visitors to the coastal public property and for residents of the Island - reduce crest height by combining with other FDM **Economic impact** ○ High maintenance cost – steel gabion mattresses corrode

The only soft engineering option evaluated is the Vegetated dune option, which can be seen in Table 6-8.

Table 6-8: Advantages and disadvantages of the vegetated dune flood defence option

Description	Advantages	Disadvantages
Vegetated dune	**Hydraulic performance** ○ Can achieve crest height to protect estuary against ocean flooding from berm overtopping ○ Can be designed to suit beach form ○ Effective in trapping windblown sand – builds sediment reservoir ○ Will limit 'beach creep' due to rising sea levels and increased storminess ○ May force more flood water around Island – increase scouring of accumulated sediment **Environmental impact** ○ Harnessing of natural processes ○ Habitat creation ○ Stops windblown sand into the estuary – less sedimentation ○ Beneficial for the whole estuary reach ○ Aesthetically pleasing **Economic impact** ○ Cost effective solution if there is no great loss of sand	**Hydraulic performance** ○ Can only be implemented partially on the beach ○ May cause mouth siltation ○ Sensitive to erosion from large storm events – mitigate by installing sleeping defence or larger dune ○ No protection against fluvial or tidal flooding **Environmental impact** ○ Uncertainty of influence on frequency of mouth condition ○ Needs to restrict access from public ○ Will maybe obstruct the ocean view of the Island residents **Economic impact** ○ High level of maintenance likely

The base condition, do-nothing option is evaluated in Table 6-9.

Table 6-9: Advantages and disadvantages of the Do-Nothing option

Description	Advantages	Disadvantages
Do-Nothing	**Hydraulic performance** ○ No interference with present hydraulic regime. **Environmental impact** ○ Estuary remains as is **Economic impact** ○ No cost of implementing a flood defence measure	**Hydraulic performance** ○ Potential for loss of life due to flooding ○ Potential for significant damage to low-lying property due to flooding **Environmental impact** **Economic impact** ○ Insurance premiums for vulnerable existing development will increase or cannot insure ○ Reparation of flood-damaged infrastructure ○ Ongoing cost of water released for emergency breaches of the estuary mouth berm

From Table 6-1 to Table 6-9 some general conclusions will be made to narrow down on potential FDM that are practical, environmentally friendly and economically feasible, as discussed in the following sections.

6.1.1 Flood defence measure in the lower estuary basin

Various FDM for direct application to the vulnerable areas of the Island were evaluated. The concrete structures, like the mass gravity concrete sea wall, L-shaped concrete sea wall and the concrete armoured dike/revetment structure were deemed to be too intrusive to incorporate into the natural setting of the estuary and will also be too visually obtrusive. The concrete options were further deemed to be too expensive for the benefit they provide. The same can be achieved with cheaper options. These concrete options will therefore not be evaluated further.

Gabion options (vertical and sloped) provide various amounts of benefits due to their flexibility. The estuary is a semi-sheltered location, where direct wave attack to the Island seldom occurs, and the mix of fresh and saline water makes it less chemically aggressive than the pure ocean environment. Gabions are known to require more maintenance than other generally more expensive FDM when subjected to a wave loading, thus increasing the lifecycle cost. Gabions are also more sensitive to direct wave attack and can only withstand up to a 1.5 m wave, which means the Gabions will likely be damaged during the design storm and will need repair. The vertical gabion sea wall option will be evaluated further for application to the vulnerable areas around the Island where a relatively high riverbank is found and where the width is constrained by the natural channel around the Island.

The dike option also became an attractive option as the maintenance is low and the constructability is relatively simple. It is also possible to incorporate a dike structure with other FDM. The dike structure may be too wide to implement around the whole of the vulnerable Island area. This can be mitigated by incorporating other FDM which can achieve a steeper seaward profile. The dike option is particularly attractive for the vulnerable low-lying area on the North-Western side of the Island, as a certain crest height can be obtained without a high-lying bank behind it, such as required for the revetment option. The dike option will be evaluated further as a separate and combination-type FDM.

The revetment option is particularly attractive for application at the areas of the Island that have a relatively high-lying river bank and that are vulnerable to direct wave attack. The rubble mound revetment type will be evaluated further as it is more aesthetically pleasing (or more natural) than the concrete armoured revetment along with the other benefits, like the wave energy dissipation and its functionality despite foundation settling outweigh the disadvantages. The concrete impervious revetment option, like the other concrete structures will not be evaluated further.

6.1.2 Flood defence measure on the beach

The vegetated dune option, where a dune is established to harness aeolian sediment transport processes to build up a dune, will be the only soft engineering option to be evaluated further. The FDM is considered an aesthetically pleasing and relatively cheap option that will keep the estuary in its natural state. The vegetated dune option will severely limit the estuary mouth berm from "creeping" inland due to rising sea levels and an increase in storminess. The increased berm height will stop overtopping of the berm from large storm waves. The concern, however, about this option, is the influence that the vegetated dune will have on the mouth condition and on open mouth frequency and on the flushing efficiency during large fluvial floods. This defence option will not be evaluated as a separate FDM, but rather only as a combined FDM with the current emergency protocol (discussed in Section 3.2.4) of establishing an emergency outflow channel to the ocean by a flush-release or breach, as the vegetated dune will only help defend against marine-driven flooding, and not against fluvial flooding.

6.2 Multi – criteria analysis

A MCA was performed to objectively identify the most attractive FDM for the Island in the lower reaches of the Great Brak estuary. The criteria used to score each FDM was determined in Section 4.7. The MCA was done for lone standing FDM and for combinations of FDM.

The MCA was performed by assigning certain weights to the evaluation criterion and then rating each FDM out of 10 for that specific criteria. 10 being a very good rating and 0 a very bad rating. The criteria weighting was then applied to the ratings to calculate the overall score out of 100 for each FDM. The Hydraulic Performance criteria carries most of the weighting (50%) to ensure that the objectives of this study are met. The Environmental Performance criteria carries the second most weight, chosen as 30% and the Economic Performance carries 20%.

The three FDM identified in Section 6.1.1 for further evaluation are the Armoured Dike, Rubble Mound Rock Revetment and the Gabion Vertical Sea Wall. These hard engineering structures are evaluated in the MCA to be applied around the entire Island as a flood defence measure. These structures are deemed to be the three best methods of preventing flood inundation of the low-lying properties on the Island. See Table 6-10 for the MCA for single option FDM.

As described in Section 6.1, some of these structures are better in dissipating wave energy than others and some are better applied for river flooding conditions. The Island does not need protection for wave attack along the whole perimeter – thus a combination of all the hard engineering options evaluated in Table 6-10 are possible, to find an optimum and cost-effective solution. The combination of these structures will also be evaluated in an MCA. The two, top-scoring FDM in Table 6-10 will be evaluated as a combination solution.

The combination of a vegetated dune option as defence from marine flooding – mainly from wave overtopping and the current emergency protocol of manipulating the mouth prior to an expected river flood as a flood defence option will also be evaluated. This option has the potential to be the most cost-effective solution considered, as the only capital costs foreseen are the engineering works, introduction of suitable vegetation, demarcating of a no-go zone and the building of timber walkways for access to the beach (if the dune impedes access to the beach) and the maintenance costs entails the upkeep of the vegetation and walkways. The uncertainty of effectiveness and influence on the estuary mouth condition are the largest concerns regarding this option. See Table 6-11 for the MCA for combination FDM.

Table 6-10: MCA for single option FDM

Criteria	Weight	Armoured Dike	Rubble mound Revetment	Gabion Vertical Seawall
Hydraulic Performance	**50**			
Protection against extreme fluvial flooding	15	8	7	7
Protection against extreme coastal flooding	15	4	8	2
Estuarine sediment flushing efficiency	10	7	5	6
Sustainability of protection measure	10	8	8	3
Environmental Performance	**30**			
Keep estuarine ecological status as close as possible to natural state	10	5	4	5
Maintain recreational potential of estuary	10	6	4	6
Benefit entire community	5	2	2	2
Accepted construction methods	5	7	7	6
Economic Performance	**20**			
Capital	15	4	3	7
Maintenance	5	7	8	2
	100	**58**	**56**	**49**

The MCA for the single option FDM identified the Armoured Dike and the Rubble Mound Revetment as the two top-scoring FDM, with the Armoured Dike identified as the preferred option. As discussed in Section 6.1, the Armoured Dike is an easily overtopped structure and will need a very high crest height for the area vulnerable to wave attack to sufficiently stop the overtopping of the large wave event. Thus, a combination FDM option of an Armoured Dike, applicable for the areas not prone to wave attack, and a Rubble Mound Revetment for the area prone to wave attack will be evaluated in Table 6-11.

The Gabion sea wall received the lowest score for protection against wave attack, the sustainability of protection measure and for maintenance cost which caused it to be the least preferred option. This is surprising as this option was deemed to be the least expensive option w.r.t the capital cost.

The MCA for the combination of solutions was performed in Table 6-11. The MCA scored the combination of the vegetated dune and emergency protocol poorly in the Hydraulic Performance criteria but very well in the other two categories. This is due to concerns regarding the hydraulic performance expressed in preceding paragraphs. The option also has room for error seeing as the emergency protocol has a predominant human decision-making element and the availability of water from the Wolwedans Dam, to assist a breaching event, is not certain. Errors like misjudging the emergency conditions or failure to breach in time might cause water to dam up even higher in the lower estuary basin, especially with restricted mouth width.

The combination MCA was interpreted to encourage combinations of the hard engineering FDM considered as the combination of an Armoured Dike and a Rubble Mound Revetment scored the highest of all the FDM considered. Thus, this combination was identified as the ultimate preferred flood defence option with the highest score in the MCA (63.5 %) and will be designed on a conceptual basis in subsequent sections.

Table 6-11: MCA for combination FDM

Criteria	Weight	Rubble mound revetment + Armoured dike	Vegetated dune + the current emergency protocol
Hydraulic performance	**50**		
Protection against extreme fluvial flooding	15	8	3
Protection against extreme coastal flooding	15	8	6
Estuarine sediment flushing efficiency	10	8	1
Sustainability of protection measure	10	9	4
Environmental performance	**30**		
Keep estuarine ecological status as close as possible to natural state	10	3	7
Maintain recreational potential of estuary	10	6	6
Benefit entire community	5	2	6
Accepted construction methods	5	5	7
Economic performance	**20**		
Capital	15	4	8
Maintenance	5	8	3
	100	**63.5**	**51.5**

6.3 Conceptual design of the preferred flood defence measures

The MCA identified a combination solution of an Armoured Dike and a Rubble Mound Rock Revetment as the ultimate preferred flood defence measure. This composite structure is designed on a conceptual basis in following sections. The conceptual design will attempt to define the geometric requirements of the proposed structures, whereas the geotechnical design aspects and detailed structural design aspects will not be addressed as it is deemed to be part of the detailed design phase.

It is proposed that the Rock Revetment structure is placed along the southwestern quadrant of the Island to protect against direct wave attack and the Armoured Dike around the rest of the Island. The average centreline length of the Revetment structure is 423.5 m and 962 m for the Dike structure. See Figure 6-1 for the conceptual plan-layout of the proposed flood defence structure.

The combined Dike and Revetment structure will fully encircle the Island. The areas where they meet will need special attention during the design phase of the project. It is recommended that, at the transitions, a degree of overlapping should be considered, to protect the core material and ultimately the integrity of the structure.

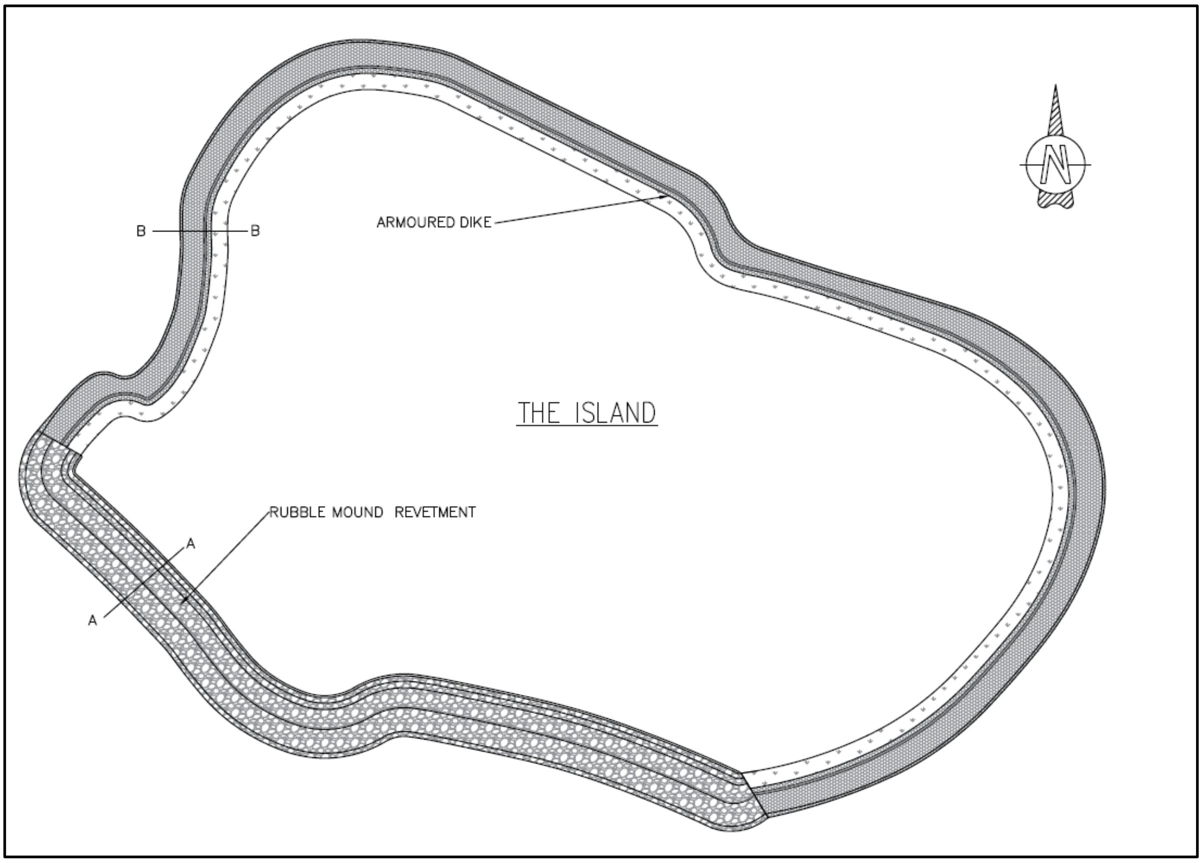


Figure 6-1: Conceptual plan-layout of proposed flood defence measure around the Island

6.3.1 Rubble mound revetment

The rubble mound, rock revetment is designed according to the shore protection project guidelines prescribed in the Rock Manual (2007) and the Coastal Engineering Manual (2006). The rubble mound revetment will serve as adequate protection against direct wave attack and will be designed for the marine design conditions described in Section 5.

The objective of the conceptual design was to offer adequate protection against the extreme waves while attempting to find the lowest crest height possible for the structure, as the flood defence measure should, as far as possible, not interfere with natural functioning, aesthetic values or the recreational potential of the estuary. Blocking the view of the residents on the Island can be viewed as a major interference to the social value of the estuary. The structure also needs to be economically viable, thus the steepest possible slope of the structure will be pursued.

6.3.1.1 Summary of design conditions

See Table 6-12 for a summary of the design conditions derived in Section 5.

Table 6-12: Summary of marine design conditions

Parameter	Year 2030				Year 2050				Year 2100			
	1:1	1:25	1:50	1:100	1:1	1:25	1:50	1:100	1:1	1:25	1:50	1:100
Extreme SWL – m MSL	1.87	2.20	2.24	2.28	2.07	2.40	2.44	2.48	2.72	3.05	3.09	3.13
Water depth - m	2.37	2.7	2.74	2.78	2.57	2.9	2.94	2.98	3.22	3.55	3.59	3.63
Design wave height at toe - m	1.53	1.82	1.87	1.91	1.62	1.91	1.96	2.01	1.92	2.21	2.26	2.31
Design wave period – s	11.9	13.1	13.3	13.6	11.9	13.1	13.3	13.6	11.9	13.1	13.3	13.6
Estuary bed level – m MSL	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5

6.3.1.2 Armour rock stability

The desired rock size and mass for stability during the design wave attack was calculated using the shallow-water formulae of Van der Meer, prescribed by CIRIA (2007). The stability-calculations were done for the various return-period wave events and for different structure lifetimes. The 1:1-, 1:25-, 1:50- and 1:100-year return-period waves were used as the design conditions combined with the calculated Extreme SWL for the year 2030, 2050 and 2100 to account for sea level rise at different periods of the century.

The wave breaking type on the structure was calculated to all be surging waves ($\xi_{s-1,0} \geq \xi_{cr}$), and thus Equation 6-1 describes Van der Meer's shallow water formulae for surging waves ($\xi_{s-1,0} \geq \xi_{cr}$):

$$\Delta D_{n50} = \frac{H_s}{c_s \cdot P^{-0.13} \cdot \left(\frac{S_d}{\sqrt{N}}\right)^{0.2} \left(\frac{H_s}{H_{2\%}}\right) \sqrt{\cot\alpha}\, (\xi_{s-1,0})^P} \qquad \textbf{6-1}$$

Where,

D_{n50} = Nominal diameter size of armour rock (m)

Δ = Relative buoyant density = $\frac{\rho_r}{\rho_w} - 1$, with $\rho_r = 2650\ kg/m^3$, $\rho_w = 1025\ kg/m^3$

H_s = Significant wave height at toe of structure (m)

c_s = 1.4 (-), $c_{pl} = 8.4\ (-)$

P = Permeability, chosen as 0.4 (double layered permeable rock armour)

S_d = Damage (1 = no damage, 2 = 0-5% damage)

N = Number of waves (calculated from storm duration)

$\xi_{s-1,0}$ = surf similarity parameter, calculated from the mean energy wave period, $T_{m-1,0}$

The permeability of the structure was found to be a significant factor in providing adequate energy dissipation, thus a permeable structure will be pursued, which consists of a double layer of armour rock, and a filter or under-layer of armour to provide drainage. The core material will be protected from washing away by means of a geotextile. The structure is designed for 0-5% damage during the design storm and wave loading. The best geometry for the revetment is an important factor as it has a direct correlation to the overall cost of the project. Steeper slopes will require larger armour stone, but will need less building material overall. The best geometry for the revetment will be the subject of analysis, with the desired crest height to be as low as possible, as not to hinder the view of the ocean for the residents on the Island. The design waves in depth-limited conditions, for the various return periods were calculated in Section 5.2.2.3, whereas the return-period extreme water level was calculated in Section 5.3. The design wave height corresponding to the depth of water at the toe of the structure (located at -0.5 m MSL), was determined via interpolation from the values presented in Table 5-7.

Three different slope angles, as well as the structure permeability, were analysed in the process of finding the best design. Table E-1 to Table E-12, in Appendix E, contains the calculated armour stone requirements for various slope angles and permeability configurations. From the tables, the impact of the slope angle and the permeability can clearly be seen. Permeable and more gradually sloped structures require far smaller armour stone than impermeable, steep-sloped structures.

See Box 6-1 for an example of the calculation procedure for the armour rock requirements for one set of input parameters.

Box 6-1: Calculation example for a single set of input parameters

Van der Meer – shallow water

- Year 2100, 100 – year Extreme SWL = +3.13 m MSL, thus depth at toe = 3.63 m
- 100 – year wave:
  - H_S = 2.31 m, T_P = 13.6 s
- Structure slope, $\tan\alpha = 0.5$
- Permeable structure – P = 0.4
- Structure damage – S = 2 (0-5%)
- Storm duration: 2 hours
- $\frac{H_s}{H_{2\%}} = 0.8$

$$T_{m-1,0} = \frac{T_P}{1.1} = 12.36\ s$$

$$L_{m-1,0} = \frac{g{T_{m-1,0}}^2}{2\pi} = 238.5\ m$$

$$\xi_{m-1,0} = \frac{\tan\alpha}{\sqrt{H_{mo_{toe}}/L_{m-1,0}}} = 5.08$$

$$\xi_{Cr} = \left[\frac{c_{pl}}{c_s} \cdot P^{0.31}\sqrt{\tan\alpha}\right]^{\frac{1}{P+0.5}} = 3.63$$

$$\xi_{m-1,0} > \xi_{Cr}\ \therefore Surging\ waves$$

Thus, Equation 6-12-22 holds:

$$\Delta D_{n50} = \frac{H_s}{c_s \cdot P^{-0.13} \cdot \left(\frac{S_d}{\sqrt{N}}\right)^{0.2}\left(\frac{H_s}{H_{2\%}}\right)\sqrt{\cot\alpha}\,(\xi_{s-1,0})^P}$$

$$D_{n50} = 0.7\ m$$

Thus, the nominal mass of the rock:

$$M_{n50} = \rho_r {D_{n50}}^3 = 906.8\ kg$$

6.3.1.3 Overtopping

An overtopping analysis of the rubble mound revetment structure was performed to find the required crest height for a chosen allowable overtopping discharge, q (m³/s/m). The critical allowable overtopping discharges for various situations and damage parameters relevant to this case are given by CIRIA (2007).

Table 6-13: Critical allowable overtopping discharges (Source: Adapted from CIRIA 2007)

Damage parameter	Critical overtopping discharge – q (m³/s/m)
No damage – Revetment sea walls	$q < 0.05$
No damage - Buildings	$q < 1\times10^{-6}$
Minor damage (to fittings etc.) – Buildings	$1\times10^{-6} < q < 3\times10^{-5}$
Structural damage to buildings	$q > 3\times10^{-5}$

The chosen design damage to the buildings in the lee of the structure will dictate the allowable overtopping discharges. The Minor Damage to Buildings (CIRIA 2007) allowable overtopping discharge will be used as the design criteria, in an attempt to keep the crest height as low as possible. The freeboard, R_C, relates to the vertical difference between the crest height of the structure and the SWL. To keep the required R_C as low as possible, a berm will be added to the face slope of the revetment. The influence of the berm is added by means of a reduction factor in the TAW (2002) method. To achieve the optimum reduction factor, 0.6, the crest berm must be located at the design SWL and be of a certain length. The optimum length of the berm can be calculated using geometric relationships described in Appendix E. The optimum berm geometry, for the three structure slopes considered, was calculated and is presented in Table E-13 to Table E-15.

The TAW (2002) method was used to calculate the required freeboard for bermed structures. See Equation 6-2 for the formulae describing the overtopping discharge for the case where $\gamma_b \xi_{m-1,0} \leq\cong 2.0$.

$$\frac{q}{\sqrt{gH_{m0}{}^3}} = \frac{A}{\sqrt{\tan\alpha}} \gamma_b \xi_m EXP\left(-B\frac{R_C}{H_{m0}\xi_{m-1,0}\gamma_b\gamma_f\gamma_\beta}\right) \tag{6-2}$$

And Equations 6-3 for $\gamma_b \xi_{m-1,0} >\approx 2.0$

$$\frac{q}{\sqrt{gH_{m0}{}^3}} = C \cdot EXP\left(-D\frac{R_C}{H_{m0}\gamma_f\gamma_\beta}\right) \tag{6-3}$$

Where;

q = overtopping discharge (m³/s/m)

A = 0.067 (-), As recommended by CIRIA (2007)

B = 4.3 (-), As recommended by CIRIA (2007)

C = 0.2 (-), As recommended by CIRIA (2007)

D = 2.3 (-), As recommended by CIRIA (2007)

$\gamma_b, \gamma_f, \gamma_\beta$ = Reduction factors for a berm ($\gamma_b = 0.6$), roughness elements (double layer rock armour, γ_f = 0.4), and oblique waves ($\gamma_\beta = 1$).

For the case of very shallow foreshores, where a large surf-similarity parameter is observed, the overtopping will be greater than calculated with Equation 6-2 and 6-3, thus for the case where $\xi_{m-1,0} > 7$, Equation 6-4, must be used. Further, for the case where: $5 < \xi_{m-1,0} < 7$, it is recommended that the overtopping discharge be calculated by means of interpolation between Equations 6-3 and 6-4.

$$\frac{q}{\sqrt{gH_{m0}{}^3}} = 0.21 \cdot EXP\left(-\frac{R_C}{H_{m0}\gamma_f\gamma_\beta\left(0.33 + 0.022\xi_{m-1,0}\right)}\right) \quad \text{6-4}$$

The influence of a crest berm was also under consideration. The overtopping discharges for the influence of a crest berm and crest wall can be calculated using Owen's method, described by Equation 6-5. The berm configuration (See Figure 6-2) and coefficients for the described crest configurations are reportedly only valid for a 1:2 slope.

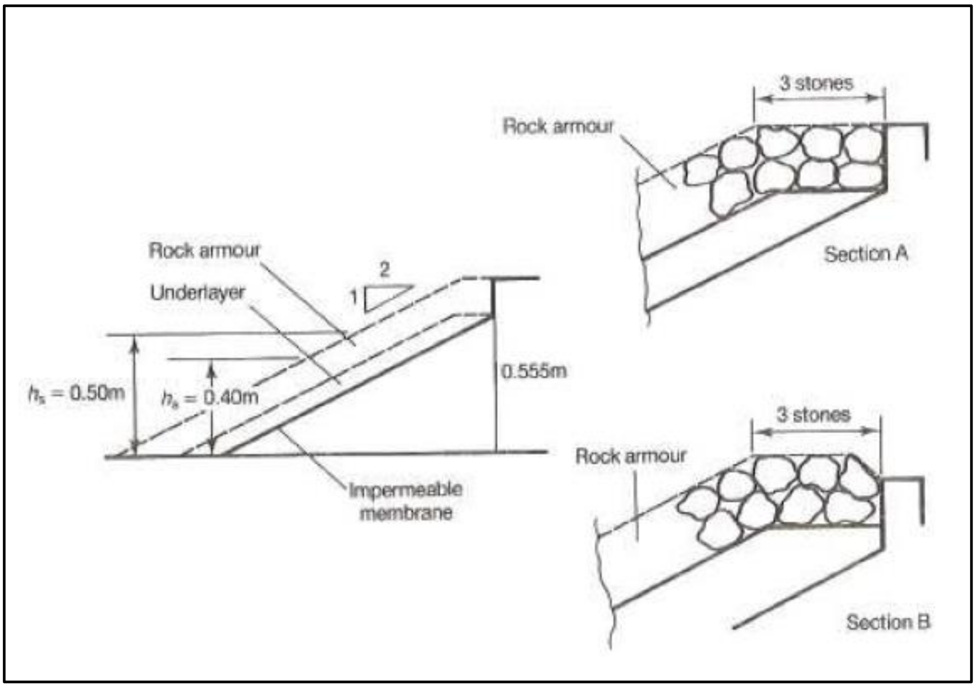


Figure 6-2: Crest berm configurations relating to Owen's method (Source: CIRIA 2007)

$$Q^* = a(F^*)^{-b} \qquad \textbf{6-5}$$

Where;

Q^* = Dimensionless specific overtopping discharge (-)

a = Coefficient (-), Section A = 3.7×10^{-10}; Section B = 1.3×10^{-9},

b = Coefficient (-), Section A = 2.92; Section B = 3.82,

F^* = $\left({R_C}/{H_S}\right)^2 \sqrt{{S_{om}}/{2\pi}}$, and $S_{om} = {2\pi H_S}/{gT_m{}^2}$

The overtopping discharge, q, is subsequently calculated by using the relationship described by Equation 6-6

$$Q^* = \frac{q}{T_m g H_s} \qquad \textbf{6-6}$$

Table 6-14 shows the calculated required freeboard, R_C, above the extreme SWL, for an overtopping discharge, q <≈ 3×10^{-5}, for minor damage to buildings in the lee of the structure. Conclusions of the geometry of the structure can be made using these values. Owen's explicit formulae for crest berms with low crest walls (Section A in Figure 6-2), yielded similar R_C values to that calculated by the TAW method for bermed structures, for the smaller wave conditions (in year 2030 and 2050). In the year 2100, the required R_C for these crest bermed structures are consistently lower than for the face slope bermed structures.

The crest berm width, as specified by Section A in Figure 6-2, is only 3*D_{n50} of the required armour stones, whereas the required face slope bermed structures with the same slope (1:2), requires much wider berms (See Table E-13 to Table E-15). Wider-bermed structures will require more construction materials and subsequently be more expensive, therefore, the crest berm structure with the slope of 1:2 will be chosen as the desired geometry for the rubble mound revetment. The required crest levels can be calculated by adding the freeboard, R_C, to the corresponding extreme SWL.

See Box 6-2 and Box 6-3 for an example of the calculation procedure followed to calculate the required freeboard, R_C, using the TAW (2002) and Owens explicit formulae for a single set of input parameters.

Table 6-14: Required freeboard, R_C (in meter above SWL), for Minor damage to buildings ($q < 3\times10^{-5}$ $m^3/s/m$), calculated for various slopes, return periods, berm configurations in different periods of the century.

	Slope	1:1	1:25	1:50	1:100	
Year 2030 Extreme Still Water Level (m MSL)		1.87	2.20	2.24	2.28	2030
TAW (2002a) – bermed structure	Slope 1:3	2.83	3.44	3.54	3.64	
	Slope 1:2	2.86	3.48	3.58	3.7	
	Slope 1:1.5	3.2	3.91	4.02	4.15	
Owen (1980) – crest berm	Section A (1:2)	2.78	3.47	3.58	3.72	
	Section B (1:2)	4.2	5.19	5.35	5.55	
Year 2050 Extreme Still Water Level (m MSL)		2.07	2.40	2.44	2.48	2050
TAW (2002a) – bermed structure	Slope 1:3	3.02	3.64	3.74	3.84	
	Slope 1:2	3.04	3.67	3.78	3.88	
	Slope 1:1.5	3.39	4.1	4.22	4.35	
Owen (1980) – crest berm	Section A (1:2)	2.92	3.63	3.74	3.89	
	Section B (1:2)	4.42	5.42	5.58	5.79	
Year 2100 Extreme Still Water Level (m MSL)		2.72	3.05	3.09	3.13	2100
TAW (2002a) – bermed structure	Slope 1:3	3.86	4.3	4.39	4.49	
	Slope 1:2	3.66	4.29	4.39	4.5	
	Slope 1:1.5	3.93	4.67	4.79	4.94	
Owen (1980) – crest berm	Section A (1:2)	3.41	4.14	4.26	4.41	
	Section B (1:2)	5.12	6.16	6.33	6.54	

Box 6-2: Calculation example for a single set of input parameters

TAW (2002a) method

- Year 2100, 100 – year Extreme SWL = +3.13 m MSL, thus depth at toe = 3.63 m
- 100 – year wave:
  - H_{m0} = 2.31 m, T_P = 13.6 s
- Structure slope, $\tan\alpha = 0.5$
- $\gamma_f = 0.4$ and $\gamma_\beta = 1$
- Coefficients, C = 0.2 and D = 2.3
- Find R_C for allowable overtopping discharge: Minor damage: $q < 3\times10^{-5}$

$$\xi_{m-1,0} = \frac{\tan\alpha}{\sqrt{H_{mo\,toe}/L_{m-1,0}}} = 5.08$$

$$\gamma_b \xi_{m-1,0} = 3.05 > 2$$

Thus, Equation 6-32-22 holds:

$$\frac{q}{\sqrt{gH_{m0}{}^3}} = C \cdot EXP\left(-D\frac{R_C}{H_{m0}\gamma_f\gamma_\beta}\right)$$

Then:

$$R_C = 4.5\ m$$

Box 6-3: Calculation example for a single set of input parameters

Owen (1980) method

- Year 2100, 100 – year Extreme SWL = +3.13 m MSL, thus depth at toe = 3.63 m
- 100 – year wave:
  - H_{m0} = 2.31 m, T_P = 13.6 s
- Structure slope, $\tan\alpha = 0.5$
- Section A: a = 3.7×10^{-10} and b = 2.92
- Find R_C for allowable overtopping discharge: Minor damage: q < 3×10^{-5}

$$S_{om} = {}^{2\pi H_S}\!/_{gT_m{}^2} = 0.0097$$

Then;

$$F^* = \left({}^{R_C}\!/_{H_S}\right)^2 \sqrt{{}^{S_{om}}\!/_{2\pi}}$$

Substituting F^* into Equation 6-5:

$$Q^* = a\left(\left({}^{R_C}\!/_{H_S}\right)^2 \sqrt{{}^{S_{om}}\!/_{2\pi}}\right)^{-b}$$

Equation 6-6 can then be written as:

$$a\left(\left({}^{R_C}\!/_{H_S}\right)^2 \sqrt{{}^{S_{om}}\!/_{2\pi}}\right)^{-b} = \frac{q}{T_m g H_s}$$

Then, solving for R_C:

$$R_C = 4.41\ m$$

6.3.1.4 Toe stability

The toe of the revetment structure is an important feature, which protects the main armour layer from scour-induced damage. The toe is constructed by creating a horizontal berm with two to three extra rows of armour rock (USACE 2006). The toe berm dimensions can be calculated from the minimum stability curve given for double-layered breakwaters in Figure E-2 in Appendix E. The stability parameter is calculated by Equation 6-7.

$$N_S = \frac{H}{\Delta D_{n50}} \qquad \textbf{6-7}$$

Where:

H = Wave height in front of breakwater (m)

Δ = ${}^{\rho_r}/_{\rho_w} - 1$

D_{n50} = Median cube length of the armour stone (m)

The stability parameter can be used to determine the ratio of water-depth above the berm, h_b, and the water-depth in front of the toe, h_s. This ratio is subsequently used to determine the required toe berm length, B, calculated as $3{*}D_{n50}$, and the height of the toe. The main armour rock median cube length, calculated for stability under wave attack by the Van der Meer formulation (Section 6.3.1.1) will be used as the input for the stability parameter. For each armour stone size, a maximum h_b/ h_s ratio of between 0.4 – 0.45 was found. See Table 6-15 for the calculated toe berm length and height.

Table 6-15: Revetment toe specifications for stability

Parameter	Year 2030				Year 2050				Year 2100			
Design storm	**1:1**	**1:25**	**1:50**	**1:100**	**1:1**	**1:25**	**1:50**	**1:100**	**1:1**	**1:25**	**1:50**	**1:100**
D_{n50} - m	0.46	0.54	0.55	0.56	0.49	0.57	0.58	0.59	0.6	0.68	0.69	0.70
Toe berm length -m	1.37	1.61	1.65	1.68	1.47	1.71	1.74	1.78	1.80	2.03	2.07	2.10
Toe berm height - m	1.42	1.62	1.53	1.67	1.44	1.62	1.65	1.67	1.93	1.99	2.01	2.03

6.3.1.5 Rock stability under current attack

The revetment structure will also be subjected to current-flow parallel to the structure during large river floods. Therefore, it is also important to ensure armour rock stability under the shear loading of fast-flowing currents. The rock stability of the revetment will be assessed using the stability formula of Escarameia and May (CIRIA 2007). The formula was chosen as it incorporates the effects of flow turbulence, which may be experienced at the edge of revetment structures and downstream of hydraulic structures. The stability formula of Escarameia and May is described by Equation 6-8. The formula will be used to check the stability of the chosen armour rock by calculating the limiting velocity for stability for each structure armour stone size.

$$D_{n50} = C_T \frac{{u_b}^2}{2g\Delta} \qquad \textbf{6-8}$$

C_T = Turbulence coefficient (-), 12.3r + 0.2 where r = 0.2 for edges of revetment in straight reaches

u_b = Near bed velocity, suggested to be 0.74-0.9 times the depth-averaged velocity, U, where no current velocity measurements are present: 0.8*U is chosen.

The critical or limiting velocity for the chosen armour stone for each structure was calculated using Equation 6-8, the results can be seen in Table 6-16. From the table, it is apparent that all the armour stone sizes will be stable in a current of between 3-4 m/s.

Table 6-16: Critical flow velocities for stability of armour stone under current attack.

Parameter	**Year 2030**				**Year 2050**				**Year 2100**			
Design storm	**1:1**	**1:25**	**1:50**	**1:100**	**1:1**	**1:25**	**1:50**	**1:100**	**1:1**	**1:25**	**1:50**	**1:100**
Critical velocity – U (m/s)	3.14	3.4	3.44	3.47	3.25	3.5	3.54	3.57	3.6	3.82	3.85	3.88
Dn_{50} - m	0.46	0.54	0.55	0.56	0.49	0.57	0.58	0.59	0.6	0.68	0.69	0.70

According to CIRIA (2007), a 4 m/s current is a typical value of observed flow velocities in estuaries. However, in the absence of local current measurements during storm flows and to avoid over-designing of the revetment, the armour stone sizes identified for stability during wave attack will be chosen as the design stone size. The 4 m/s current, as suggested by CIRIA (2007), is a conservative value when compared to the only available current velocity measurement taken during a breaching event at Great Brak. The highest current velocity measured at the mouth of the estuary during a breaching event was observed to be approximately 1.28 m/s (Beck, *et al* 2004). The current velocity at the revetment structure will likely not reach the flow velocity observed at the mouth.

It is possible to construct a form of river-training structure, like a spur-dike, upstream of the revetment, to ensure that the flow velocity at the edge of the revetment does not surpass the critical flow velocities of the armour rock. The armour rock size can also be increased to ensure stability, however, this is not the preferred option as it would need a larger initial economic input as the volume of the required armour rock will increase. The availability of large armour stone and its proximity to the site will also be significant factors to consider.

6.3.1.6 Final concept cross-sectional layout

From the preceding sections, the cross-section of the rubble mound revetment structure can be designed. The structure is chosen to be permeable, and armoured by a double layer of armour rock. The armour rock will rest on an under-layer. The under-layer functions to provide drainage to the structure. The core material will be protected from wash-away by a geotextile, which is to be placed on the core material underneath the under-layer. See Table 6-17 for the layer specifications of the revetment structure.

Table 6-17: Rubble mound layer specifications for permeability parameter of 0.4 (Source: USACE 2006)

Layer specifications	**Nominal cube length: D_{n50}**	**Layer thickness**
Armour layer	${D_{n50}}^{armour}$	$2\times{D_{n50}}^{armour}$
Under-layer:	$0.5\times{D_{n50}}^{armour}$	$1.5\times{D_{n50}}^{armour}$
Core	$0.25\times{D_{n50}}^{underlayer}$	–

The required crest height of the structure was calculated by analysing various slopes and geometric configurations for a chosen allowable overtopping rate. The allowable overtopping rate for “Minor Damage to buildings” in the lee of the structure was chosen as a reasonable overtopping rate. The revetment structure will not receive any damage during the design storm and the required crest height is lower than for the “No Damage to buildings” criteria. The structure slope of 1:2 and a crest berm with the width, 3*D_{n50}, was identified as the optimum structure for all design storms and lifetimes and was chosen as the preferred structure geometry. The structure was only designed for a 2-hour storm duration as the extreme still water level is deemed not to be elevated at the design values for long period and the enclosed nature of estuaries will protect the Island from the full duration of a storm.

The availability of suitable armour rock is an important issue to consider during the design phase of the project as the transportation costs of the material will have a significant economic impact on the project. A source for the armour rock close to the site is preferred. Storm water drainage through the retaining wall of the revetment structure will also have to be considered in the design phase – pipes with one-way valves through the structure can achieve adequate drainage of storm water run-off from the Island. Alternative to the concrete retaining wall, sheet-piling can also be considered in the design phase of the project, to provide adequate stability to the revetment structure.

See Table 6-18 for the design specifications of the revetment structure for various design storms at various periods of the century. Figure 6-3 shows a typical cross-sectional layout of the revetment structure, design for the 1-year wave in the year 2050. The crest height of this structure can be seen to be +4.99 m MSL, which is very high and a product of conservative design assumptions. Further work may attempt to refine the design conditions to lower the required crest height.

The cross-sections designed for all the design conditions (1:1, 1:25, 1:50 and 1:100) and considered lifetimes (2030, 2050, 2100) can be seen in Appendix E.

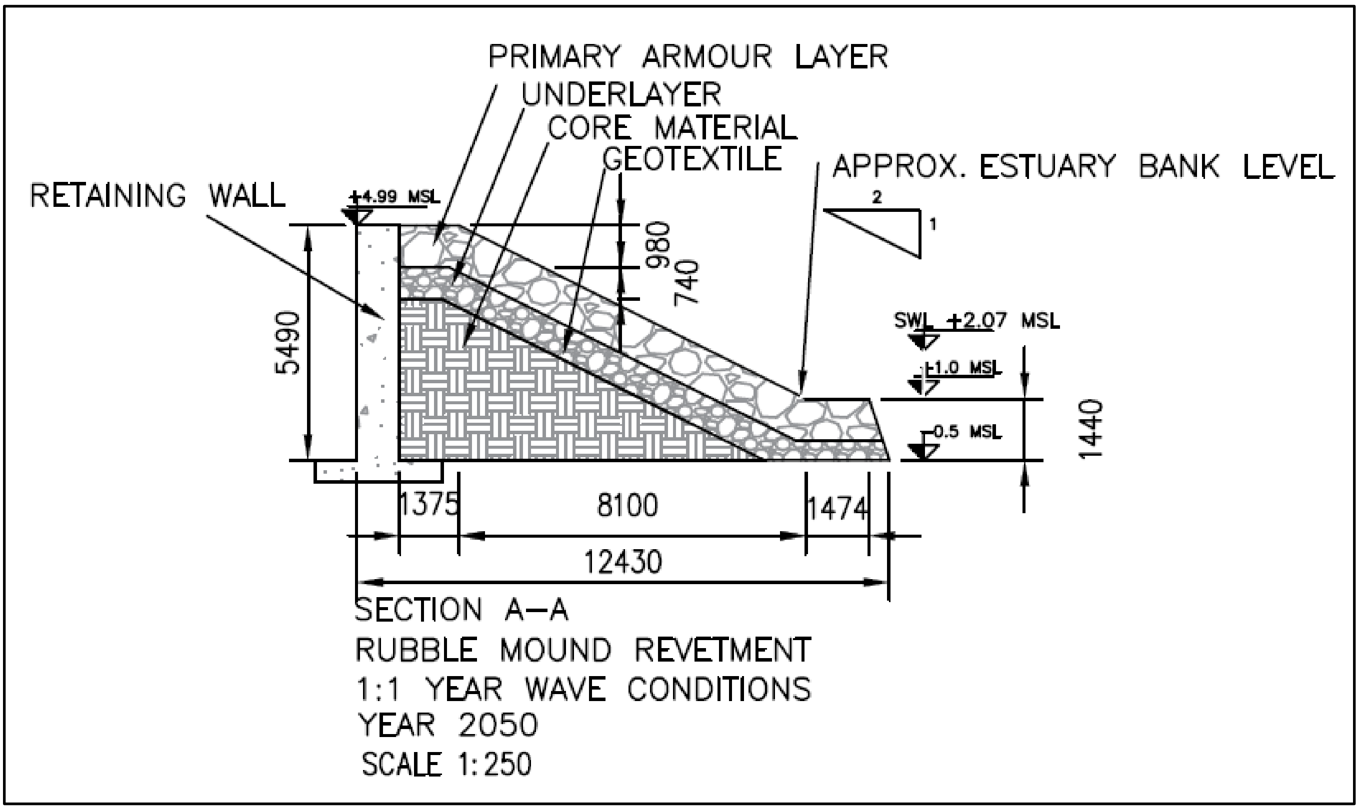


Figure 6-3: Conceptual cross-section layout of the rubble mound revetment designed for the 1-year conditions in the year 2050

Table 6-18: Concept design specifications for the armouring and geometry of the rubble mound revetment with a face slope of 1:2

Parameter	Year 2030				Year 2050				Year 2100			
Design storm	**1:1**	**1:25**	**1:50**	**1:100**	**1:1**	**1:25**	**1:50**	**1:100**	**1:1**	**1:25**	**1:50**	**1:100**
Design water depth at toe (m)	2.37	2.7	2.74	2.78	2.57	2.9	2.94	2.98	3.22	3.55	3.59	3.63
Design wave height at toe (m)	1.53	1.82	1.87	1.91	1.62	1.91	1.96	2.01	1.92	2.21	2.26	2.3
Permeability – P	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4
Damage parameter – S_d	2	2	2	2	2	2	2	2	2	2	2	2
Design storm duration (hrs)	2	2	2	2	2	2	2	2	2	2	2	2
Armour layer specifications												
Dn_{50} - m	0.46	0.54	0.55	0.56	0.49	0.57	0.58	0.59	0.6	0.68	0.69	0.7
Mn_{50} - kg	255	411	439	465	314	491	521	551	576	827	869	907
Thickness - m	0.92	1.07	1.1	1.12	0.98	1.14	1.16	1.18	1.2	1.36	1.38	1.4
Under-layer specifications												
Dn_{50} - m	0.23	0.27	0.27	0.28	0.25	0.57	0.29	0.3	0.3	0.34	0.34	0.35
Mn_{50} - kg	32	51	55	58	39	491	65	69	72	103	109	113
Thickness - m	0.69	0.81	0.82	0.84	0.74	1.14	0.87	0.89	0.9	1.02	1.03	1.05
Core material specifications												
Dn_{50} - m	0.06	0.07	0.07	0.07	0.06	0.07	0.07	0.07	0.08	0.08	0.09	0.09
Mn_{50} - kg	0.5	0.8	0.86	0.91	0.61	0.96	1.02	1.08	1.13	1.61	1.7	1.77
Geometry												
Crest height – m MSL	4.65	5.67	5.82	6	4.99	6.03	6.18	6.37	6.13	7.19	7.35	7.54
Freeboard – R_C (m SWL)	2.78	3.47	3.58	3.72	2.92	3.63	3.74	3.89	3.41	4.14	4.26	4.41
Crest berm length (m)	1.37	1.61	1.65	1.68	1.47	1.71	1.75	1.78	1.8	2.03	2.07	2.1

6.3.2 Armoured dike

The armoured dike was designed for the crest heights of +3.0, +3.5, +4.0 and +4.5 m to cater for various potential extreme water levels in the lower estuary basin. A typical cross-section will be developed for each crest height and a cost-estimate will be made for each crest height. The main hydraulic loading to design for will be current-flow that may potentially cause scouring of the core material. In estuaries, a typical current flow velocity of 4 m/s can be assumed (CIRIA 2007). For scour protection and for added depth of cut-off to limit seepage, the dike toe will be placed at -0.5 m MSL while the estuary bank is between +0.5 and +1.0 m MSL at the dike location.

Dike structures in river engineering are normally used for river entrainment caused by a temporary rise in water level and therefore, a homogenous cross-section of compacted and pervious material will be sufficient. The proposed dike is located in the lower estuary basin, partially under the high-water mark and will have a varying upstream water head, as the estuary regularly experiences water levels between +1.0 m MSL and +2.0 m MSL. Thus, the dike will be designed as a small embankment dam according to the guidelines provided in (USBR 1987) and (Novak *et al.* 2007). The dike will consist of an (1) armour unit, to protect the structure from scour induced by current flow, (2) impermeable earthen core, consisting out of a high percentage clay mixture and (3) supporting shoulders of coarser earth material to provide structural stability.

6.3.2.1 Gabion mattresses

Gabion mattresses are chosen to be the armouring unit for the dike, as it was deemed to be more aesthetically pleasing than concrete or asphalt. Smaller armour rock will also be needed to fill the gabions than conventional loose stone. The gabion mattresses can also be made to accommodate access over the dike to the estuary with built-in stairs or walkways, which may be harder to achieve with conventional armour stone.

Being in an estuary, the dike will experience secondary waves and a rapid rise and drop in water level. The minimum gabion mattress thickness required for protection against typical secondary waves was calculated for three slope configurations, i.e. 1:3, 1:2 and 1:1.5 (see Table 6-19). The steepest possible slope will be pursued as it will require less construction materials and subsequently be more economical. The effect of wind waves in that portion of the estuary basin will be neglected as no significant fetch is present for wind wave generation. Secondary waves, however will be accounted for in the design. According to CIRIA (2007), the minimum thickness of a protective gabion mattress for a wave loading, can be calculated using Equation 6-9 for the case $\cot\alpha \leq 3$. The same slopes than the revetment, i.e. 1:3, 1:2 and 1:1.5 will be under analysis.

$$t_{min} = \frac{H}{2\Delta(1 - n_v)\cot\alpha} \qquad \textbf{6-9}$$

Where:

t_{min} = minimum thickness for gabion defence under wave loading (m)

H = wave height, chosen as 1.0 m for secondary waves in estuaries (m)

n_v = porosity of revetment material, typical value of 0.35 (CIRIA 2007)

Table 6-19: Minimum thickness of gabion mattress under wave loading

Parameter	Slope - 1:3	Slope – 1:2	Slope – 1:1.5
t_{min} - m	0.16	0.24	0.32

Table E-17 shows that a mattress thickness of 0.3 m offers adequate protection against current velocities of up to 5 m/s. The minimum mattress thickness for a secondary wave height of 1 m was calculated as 0.24 m for a 1:2 slope and 0.32 for a 1:1.5 slope. Gabion mattress prices were obtained for a maximum thickness of 0.3 m and will be the limiting factor to this design. A slope of 1:2 and a mattress thickness of 0.3 m will subsequently be chosen for the development of the typical cross-sections.

For added safety, a nominal stone size of D_{n50} = 125 mm is chosen. Due to the location of the dike, where a tidal intrusion is expected, the wire mattresses will need to be galvanised and coated with PVC as corrosion protection. A geotextile will be placed under the gabion mattresses to protect the embankment material from erosion due to piping.

6.3.2.2 Impermeable core

An embankment dam needs a core of less permeable material to reduce the seepage flow of water through and underneath the structure. This will sufficiently protect the structure from piping or blowout failures, which can ultimately lead to structure failure. The less permeable material will slow the water seepage through and underneath the dam significantly, but a form of drainage still needs to be provided on the downstream side of the structure. The dike structure, like previously stressed, will not be subject to direct wave attack, thus the impermeable core should have no effect on the crest level of the dike, w.r.t wave run-up elevations.

Zoned embankment dams are recommended for small farm dams, where low cost is generally the dominant consideration (Novak *et al.* 2007). A zoned embankment dam structure, where the impermeable core is created with a high percentage clay-like soil, will form the basis of design for the proposed dike structure. The dam zoning will be relatively simple, to aid the construction process. The impermeable core will consist of rolled clay and the shoulder material of compacted earthen backfill material. A drainage blanket, consisting of coarse rock, will be added on the downstream side, to provide relief from the seepage forces.

The clay core is recommended to be as wide as economically possible, for added stability of the structure. The foundation material plays a major role in the geometry of the clay core. A more pervious foundation will require a wider core to limit seepage. Added features of the core, like a cut-off trench or a vertical diaphragm that extends further into the foundation can reduce the required width of the impermeable core. Due to limited information on the foundation material, the assumption will be made that the bedrock is relatively deep and that the in-situ sand foundation has no cohesive properties, i.e. pervious.

Design guidance on the geometry requirements of the clay core is given in Figure E-3 in Appendix E. The minimum base width of a core on impervious foundation or on shallow pervious foundations with a cut-off trench is shown by core A (Appendix E), the minimum core width for dams on deep pervious foundations without a cut-off trench is shown by core B (Appendix E). The minimum core base width for both cases, and for all dike crest elevations are shown in Table 6-20.

Table 6-20: Minimum core base width for case A and B in Figure E-3

Core base width	**Dike @ +3 m MSL**	**Dike @ +3.5 m MSL**	**Dike @ +4 m MSL**	**Dike @ +4.5 m MSL**
Minimum core A - m	3.5	4	4.5	5
Minimum core B - m	10.5	12	13.5	15

Thus, with the addition of a cut-off trench, the base width will be chosen to be slightly wider than the minimum width recommended for core A, as the assumption of a relatively deep bedrock at the site will require a wider core. The cut-off trench is designed as per the guidelines stipulated in USBR (1987). The relationship between the bottom width of the trench, the upstream water head above ground level and the depth of penetration of the trench below the ground level is given by Equation 6-10.

$$w = h - d \qquad \textbf{6-10}$$

Where:

w = bottom width of trench (m)

h = upstream water head above ground level (m)

d = depth of cut-off trench below ground level. (m)

The depth of cut-off will be limited to 0.5 m below ground level. The “ground” level will be taken as the bottom of the dike structure, which is placed at -0.5 m MSL, thus the cut-off will extend to -1 m MSL. The cut-off trench dimensions are shown in Table 6-21.

Table 6-21: Cut-off trench dimensions

Cut-off trench base width and depth	**Dike @ +3 m MSL**	**Dike @ +3.5 m MSL**	**Dike @ +4 m MSL**	**Dike @ +4.5 m MSL**
Depth of cut-off trench (m)	0.5	0.5	0.5	0.5
Upstream head (m)	3.5	4	4.5	5
Bottom width (m)	3	3.5	4	5

6.3.2.3 *Final concept cross-sectional layout*

The dike slope, armour unit- and impermeable core dimensions have been determined in the preceding paragraphs. The crest width and the drainage elements on the downstream side will be discussed in this section. The detailed geotechnical considerations like settlement and seepage flow analysis will not be accounted for in this conceptual design, but should be considered in the detailed design phase and based on measured information of the available materials and foundation material.

Novak et el. (2007) provides guidance of the minimum crest width for the zoned embankment type dike structure. A minimum width of 3 m is prescribed, for ease of access for construction vehicles. A minimum freeboard of 1 m is also prescribed for the dike to limit overtopping, which effectively causes the maximum design flood water level to be 1 m less than the crest height. The decision was therefore made to provide gabion mattress cover that extends past the crest and partially down the slope of the downstream face, to protect the shoulder from erosion caused by overtopping. This feature will be crucial, particularly for the lower crest level structures. The core of the dike structure is designed to be impermeable, which is achieved by a clay core with a cut-off trench, to limit seepage through the structure. Alternatively, the same effect can be achieved by using sheet-piling to create an impermeable core for the structure.

A horizontal drainage blanket, located under the downstream shoulder will be provided as per the guidance of USBR (1987). The drainage blanket can be created by using pervious material like sand, gravel, rock or a mixture of the material and will drain the excess water in the dike structure. Additional storm water drainage elements will also be needed to drain the run-off from the Island. This can be achieved by using pipes with one-way valves. See Figure 6-4 for the final cross-sectional layout of the armoured dike structure.

Like the revetment structure, the toe of the structure will be located at -0.5 m MSL for added scour protection. The dike and the revetment will together fully encircle the Island at the areas where they meet, they must overlap to some degree, to protect the core material at the transitions. See Figure 6-4 for the conceptual cross-sectional layout and dimensions of the dike structure with the crest height of +3 m MSL. The dimensions and cross-sections of the armoured dikes with the crest heights of +3.0,

+3.5, +4.0 and +4.5 m MSL can be seen in Appendix E. Figure 6-1 shows the conceptual plan layout of the proposed solution. See Figure 6-5 for the conceptual plan layout of the combination solution of the +3 m MSL Armoured Dike and the Revetment structure designed for the 1-year conditions in the year 2050

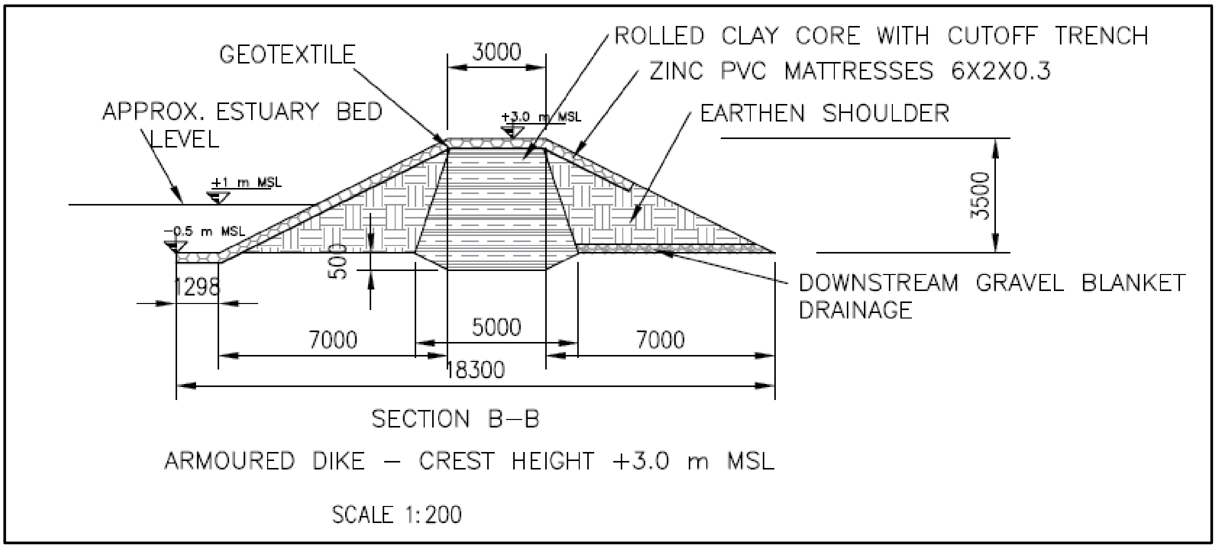


Figure 6-4: Conceptual cross-section layout of the Armoured dike structure with the crest height of +3 m MSL

Figure 6-5: Plan layout of the combination solution of the +3 m MSL Armoured Dike and the Revetment structure designed for the 1-year conditions in the year 2050.

6.4 Overtopping analysis

The chosen design conditions and design lifetimes are important factors which greatly affect the cost of the proposed project. The structures designed for the longer lifetimes will be significantly more expensive, which can make it hard to justify economically. The cheaper options, i.e. structures designed for a shorter lifetime and smaller return-period, can be pursued, but it is important to understand the consequences of the choice. Insight into the probability of occurrence of an extreme event with a return period, T, in a certain number of years, N (say design lifetime of the structure), can be gained by using Equation 6-11 (Chadwick *et al.* 2013).

$$P(X > x)_N = 1 - \left(1 - \frac{1}{T}\right)^N \qquad \textbf{6-11}$$

See Table 6-22 for the calculated probability of occurrence of the various return-period storms in the considered lifetime of the projects. The probability of the 100-year storm occurring within the next 13 years (2030) is 12%, 28% within the next 33 years (2050) and 57% within the next 83 years (2100).

Table 6-22: Design life probability of occurrence

	Return periods			
Design life	**1**	**25**	**50**	**100**
Year 2030	100%	41%	23%	12%
Year 2050	100%	74%	49%	28%
Year 2100	100%	97%	81%	57%

The revetment structures, designed in Section 6.3.1, will be subjected to an overtopping analysis where each structure will be subjected to all the extreme storm conditions considered. Equations 6-5 and 6-6 were used to estimate the overtopping discharges. The design crest height of each structure was used to determine the freeboard, R_C, during each storm condition, which was used in the calculations. The structures were designed for an allowable overtopping unit discharge of q <≈ $3{\times}10^{-5}$ m³/s/m, which correlates to minor damage to buildings. If the structure were to allow an overtopping unit discharge, q > $3{\times}10^{-5}$ m³/s/m, the buildings in the lee could experience structural damage (as per Table 6-13) and the revetment structure may fail to live up to its design criteria of protecting the buildings in its lee.

Table 6-23 contains the overtopping unit discharges, q, of each structure subjected to each of the considered wave loadings. The cells marked in red are the structures that fail to limit the overtopping within allowable rates under the future conditions. This analysis shows that the structures designed for the 2030 conditions will not offer much protection against the wave loadings considered for 2050 and

2100 conditions, save for the 1:1-year condition in 2050, where the larger structures designed for 2030 will still offer adequate protection. The structure designed for the 1:1 year conditions in 2050 will only offer protection against the same loading in 2030, and will not offer protection against larger wave loadings in 2030. As expected, the structures designed for the 2100 conditions, will offer adequate protection against most wave loadings in 2030 and 2050, save for the 1:1-year structure, which will fail to offer adequate protection against the 1:50 and 1:100 year loadings in year 2050. It is interesting to note that all the revetment structures will limit the overtopping rate so as to not experience damage.

A similar overtopping analysis cannot be performed for the armoured dike structure as return-period water levels due to fluvial floods were not determined in this study. The implications of the water level exceeding the design crest height of the armoured dike can however be investigated and discussed. Fluvial floods will cause a momentary maximum water level, which will cause overtopping of the structure if this extreme water level exceeds the design crest level. The fluvial flood will proceed to flow into the ocean and the water level will subsequently subside. The amount of water overtopping the structure is dependent on the duration that the water level exceeds the crest level. In the 27-year recording period (1990 – 2017), a +2.9 m MSL water level, caused by a 10-year flood coinciding with a closed mouth, was recorded as the highest water level. Even larger floods are expected to occur in the next century, with increasing frequency, thus the possibility of a larger extreme flood coinciding with a full dam and closed mouth condition will increase. This may likely cause record water levels and inundation to low-lying properties.

It will be recommended that the dike structure must be built as high as economically possible and socially acceptable. The risk of overtopping, especially the lower crest height structures, should be mitigated by adequate downstream protection against erosion and drainage schemes. The temporary nature of extreme water levels in rivers and the impermeable core for seepage control will protect the structure from failures occurring due to excess pore water pressures and seepage discharges. The emergency protocol of preparing the emergency channel to allow flood waters to flow into the ocean will remain relevant if the estuary mouth berm is surveyed to be within 0.5 – 1.0 m of the crest height of the structure.

Table 6-23: Overtopping discharge, q (m³/s/m), for the various revetment structures under the various considered extreme storm conditions

		Structure											
		Year 2030				Year 2050				Year 2100			
Storm conditions		1:1	1:25	1:50	1:100	1:1	1:25	1:50	1:100	1:1	1:25	1:50	1:100
Year 2030	1:1	2.9E-05	4.7E-06	3.7E-06	2.9E-06	1.5E-05	2.8E-06	2.2E-06	1.7E-06	2.4E-06	6.5E-07	5.5E-07	4.5E-07
	1:25	2.2E-04	2.9E-05	2.3E-05	1.7E-05	1.1E-04	1.7E-05	1.3E-05	1.0E-05	1.4E-05	3.5E-06	2.9E-06	2.4E-06
	1:50	3.0E-04	3.9E-05	3.0E-05	2.3E-05	1.4E-04	2.2E-05	1.7E-05	1.3E-05	1.9E-05	4.5E-06	3.8E-06	3.0E-06
	1:100	4.1E-04	5.1E-05	3.9E-05	2.9E-05	1.9E-04	2.8E-05	2.2E-05	1.7E-05	2.4E-05	5.8E-06	4.8E-06	3.9E-06
Year 2050	1:1	6.1E-05	8.7E-06	6.9E-06	5.2E-06	3.0E-05	5.0E-06	4.0E-06	3.1E-06	4.3E-06	1.1E-06	9.3E-07	7.6E-07
	1:25	4.8E-04	5.4E-05	4.2E-05	3.1E-05	2.1E-04	2.9E-05	2.3E-05	1.7E-05	2.5E-05	5.8E-06	4.8E-06	3.8E-06
	1:50	6.5E-04	7.1E-05	5.4E-05	4.0E-05	2.8E-04	3.8E-05	3.0E-05	2.2E-05	3.2E-05	7.4E-06	6.1E-06	4.9E-06
	1:100	9.0E-04	9.5E-05	7.3E-05	5.3E-05	3.9E-04	5.1E-05	4.0E-05	3.0E-05	4.3E-05	9.8E-06	8.0E-06	6.4E-06
Year 2100	1:1	8.3E-04	7.0E-05	5.2E-05	3.7E-05	3.2E-04	3.5E-05	2.7E-05	2.0E-05	3.0E-05	6.1E-06	5.0E-06	4.0E-06
	1:25	7.7E-03	4.3E-04	3.1E-04	2.2E-04	2.5E-03	2.0E-04	1.5E-04	1.1E-04	1.7E-04	3.0E-05	2.4E-05	1.9E-05
	1:50	1.1E-02	5.7E-04	4.1E-04	2.8E-04	3.4E-03	2.6E-04	2.0E-04	1.4E-04	2.2E-04	3.8E-05	3.0E-05	2.3E-05
	1:100	1.5E-02	7.4E-04	5.3E-04	3.6E-04	4.6E-03	3.4E-04	2.6E-04	1.8E-04	2.8E-04	4.8E-05	3.8E-05	3.0E-05

6.5 Order of magnitude cost estimate

Based on the conceptual cross-sections and combined plan layout developed for flood alleviation for the Island in Section 6.3, the quantities of required construction material can be calculated. From construction material rates obtained from Roux (2017) an order of magnitude cost estimate will be derived to conclude the study on a feasible measure for flood defence. The material rates can be seen in Appendix E, Table E-18.

6.5.1 Structure cost

In order to calculate a realistic cost estimate for the flood defence measures, further assumptions must be made regarding the excavation volume and the retaining wall. The reinforced concrete retaining wall is an important structural component to the rubble mound revetment and will be a significant factor in the overall cost of the project. In this study, the detailed design of the retaining wall is deemed to be outside the scope of work, thus a conservative assumption will be made regarding the cross-sectional area of the retaining wall. A nominal thickness of 1 m, with a foundation of 2 m by 0.5 m will be assumed in the calculation of the quantities. The required excavation volume will be calculated assuming the average estuary bed level is at +1 m MSL based on estimated from the available topographic surveys. See Table 6-24 for a summary of the required quantities of the various required construction materials.

From Table 6-24 and Table E-18 the cost estimates can be made for the various structural components. The estimate will include the additional fees for professional consulting and geological surveys as a percentage of the structure cost. An Environmental Impact Assessment must be conducted as the structure will directly be located under the estuary high water mark. Thus, the fees relevant to an EIA will be included in the cost estimate of construction. From Table E-18, an EIA would cost about R300 000.

See Table 6-25 for the order of magnitude cost estimate for the Rubble mound rock revetment structure and for the Armoured dike structure. The proposed flood defence measure consists out of both the revetment and dike structure, so the total structure cost will be the sum of the chosen revetment and armoured dike structure. For example, the cost of the flood defence structure consisting of the revetment structure designed for the 1:1-year storm conditions in the year 2050 and the armoured dike structure with the crest height of +4.0 m MSL is calculated to be R47.6 million. The cheapest option was the combined revetment structure, designed for the 1:1-year storm in year 2030, and the +3 m MSL crest level dike structure with a cost of R41.7 million, without the construction and EIA cost.

Table 6-24: Table of required quantities for construction material

Parameter	Year 2030				Year 2050				Year 2100				Armoured dike			
Design storm	**1:1**	**1:25**	**1:50**	**1:100**	**1:1**	**1:25**	**1:50**	**1:100**	**1:1**	**1:25**	**1:50**	**1:100**	**3**	**3.5**	**4**	**4.5**
Length of structure – measured along centreline	418	422	423	423	420	424	424	425	423	426	427	427	952.6	958.2	966	970
Required Armour stone area	9.3	14.6	15.6	16.2	11.8	16.6	17.4	18.2	17.3	23.1	24	25	4.5	4.9	5.2	5.3
Require no. of gabion mattresses	-	-	-	-	-	-	-	-	-	-	-	-	1195	1291	1392	1440
Required under-layer area	6.7	10.5	11.8	11.9	8.4	11.6	12	13.1	12.7	16.3	16.8	17.6	-	-	-	-
Required core material area	12.3	23.5	23.6	25.6	17.9	26.1	27.5	28.7	26.3	36.1	37.9	39.7	-	-	-	-
Retaining wall area – reinforced concrete	6.2	7.7	7.8	8	7	8	8.2	8.4	8.1	9.2	9.4	9.5	-	-	-	-
Required earth fill for shoulders	-	-	-	-	-	-	-	-	-	-	-	-	16.7	21.4	26.6	32.2
Required impermeable clay core area	-	-	-	-	-	-	-	-	-	-	-	-	14.6	19	23.5	28.6
Drainage blanket - rocks	-	-	-	-	-	-	-	-	-	-	-	-	1.7	1.9	2	2.2
Geotextile length – m	10.4	13.9	14.4	14.7	12.3	14.8	15.2	15.6	14.7	17.4	17.8	18.2	15	16.1	17.2	18.3
Required approximate excavation	19	23.8	24.5	24.8	21.6	25.13	25.7	26.2	25	28.7	29.2	29.8	30	33.6	36.7	40.1

Table 6-25: Order of magnitude cost estimate for all structures (in million Rand)

	Parameter	Year 2030				Year 2050				Year 2100				Armoured dike			
	Design storm/Crest level	1:1	1:25	1:50	1:100	1:1	1:25	1:50	1:100	1:1	1:25	1:50	1:100	3	3.5	4	4.5
Revetment	Armour layer	3.51	5.53	5.92	6.16	4.47	6.32	6.64	7	6.57	8.87	9.22	9.6	-	-	-	-
	Under layer	2.25	3.53	3.98	4.01	2.82	3.92	4.07	4.4	4.3	5.56	5.73	6.01	-	-	-	-
	Core	4.36	8.42	8.46	9.2	6.38	9.41	9.91	10.4	9.47	13.07	13.73	14.43	-	-	-	-
	Retaining wall	9	11.33	11.56	11.84	10.27	11.91	12.14	12.4	12.03	13.71	13.97	14.26	-	-	-	-
Dike	Zinc PVC mattress 6x2x0.3	-	-	-	-	-	-	-	-	-	-	-	-	3.22	3.48	3.75	3.88
	Rock fill (Supply and place)	-	-	-	-	-	-	-	-	-	-	-	-	3.66	3.95	4.26	4.41
	Backfill - shoulders	-	-	-	-	-	-	-	-	-	-	-	-	1.19	1.54	1.93	2.35
	Clay - core	-	-	-	-	-	-	-	-	-	-	-	-	1.67	2.19	2.72	3.33
	Gravel drain blanket	-	-	-	-	-	-	-	-	-	-	-	-	1.31	1.44	1.57	1.72
	Geotextile	0.22	0.29	0.3	0.31	0.26	0.31	0.32	0.33	0.31	0.37	0.38	0.39	0.71	0.77	0.83	0.89
	Excavation	0.99	1.26	1.3	1.31	1.14	1.33	1.36	1.4	1.32	1.53	1.56	1.59	3.57	4.02	4.43	4.86
	Total structure cost	20.33	30.35	31.53	32.83	25.33	33.2	34.43	35.9	34.01	43.11	44.59	46.29	15.33	17.39	19.49	21.44
	Design fees	2.44	3.64	3.78	3.94	3.04	3.98	4.13	4.3	4.08	5.17	5.35	5.55	1.84	2.09	2.34	2.57
	Ground surveys	1.02	1.52	1.58	1.64	1.27	1.66	1.72	1.8	1.7	2.16	2.23	2.31	0.77	0.87	0.97	1.07
	Total cost	**23.78**	**35.51**	**36.9**	**38.41**	**29.64**	**38.84**	**40.29**	**42.1**	**39.79**	**50.44**	**52.17**	**54.16**	**17.94**	**20.35**	**22.8**	**25.08**

6.5.2 Project cost

The cost estimate, with respect to the materials, for the various structures can be seen in Table 6-25. To assess the feasibility of the proposed solution, the overall project cost must be estimated. To do so, the cost of construction needs to be estimated. The cost of construction is project-specific, as the cost of labour and transport costs for materials are largely dependent on the contractor, and until the project is placed on tender, this cost cannot be accurately calculated. For this study, to obtain a first estimate of project cost, a reasonable assumption of the construction cost will be made.

Based on previous coastal structure projects, the assumption can be made that the construction cost of a project is roughly equal to the cost of the materials (Theron pers. com 2017). This assumption is seen as conservative, and amounts to a rough estimate of the overall project cost. Preliminary and General items (which normally amounts to 30% of the cost) is assumed to be included in this assumption. Based on this assumption, the project cost for all structure combinations can be calculated, see Table 6-26. The cost of an EIA (R300 000) is included in the estimation for each structure.

The cost of maintenance for the gabion mattresses will be neglected in this project cost, as the gabion mattresses will be protected from wave attack. If the wire casings are adequately protected from corrosive and abrasive forces, as intended in this design, then the mattresses can reportedly have similar lifetimes to concrete structures (CIRIA 2007).

Table 6-26: Project cost (in million Rand) for all structure combinations under consideration

	Year 2030				**Year 2050**				**Year 2100**			
Armoured dike:	**1:1**	**1:25**	**1:50**	**1:100**	**1:1**	**1:25**	**1:50**	**1:100**	**1:1**	**1:25**	**1:50**	**1:100**
3 m MSL	84	107	110	113	95	114	117	120	116	137	141	145
3.5 m MSL	89	112	115	118	100	119	122	125	121	142	145	149
4 m MSL	93	117	120	123	105	124	126	130	125	147	150	154
4.5 m MSL	98	121	124	127	110	128	131	135	130	151	155	159

The estimated project cost will be compared to the costs estimated during the Do-Nothing option, where the Estuary Mouth Management Protocol dictates the emergency breaching of the mouth, to prevent extensive flooding of the Island properties. The cost of water is identified as the main expenditure during the Do-Nothing alternative, seeing as the water-release assisted breaching requires a fair amount of water, as highlighted in Section 3.2. The potential “savings” in terms of flood damages prevented and reduced insurance rates are not included in this analysis.

6.5.3 Project cost versus cost of water

The costs incurred during the Do-Nothing alternative is deemed to be equal to the cost of water, released out of the Wolwedans Dam for the assisted breaching of the estuary mouth when the emergency protocol is invoked. When a flood defence measure is implemented, the need for assisted emergency breaches will decline as the water level in the lower estuary basin will be able to rise above +2.2 m MSL. The assumption is that the Municipality will “save” the money dedicated to these water releases.

Table B-1, in Appendix B, was used to estimate the amount of water, on average (per year), used for performing emergency breaches. All breaching events labelled as “Emergency” or that fell outside of the period earmarked for keeping the mouth open, i.e. 1 September – 28/29 February, were included in the estimate. From 1990 to 2016, a total of 9 106 527 m³ of water was used for emergency breaches, which amounts to an average of 350 251 m³ per year. This yearly average will be used to extrapolate the cumulative cost of water over the same lifetimes as the proposed structures.

Water tariffs listed under the special tariffs for Mossel Bay Municipality Departmental usage will be used to convert the amount of water to the equivalent monetary value. The tariff for 2017 is given as R7.88/m³ and R8.35/m³ for the 2018 year (Mossel Bay Municipality 2017). The value for the 2018 year will be used as a constant as it was deemed to be a more conservative approach. It is expected that the

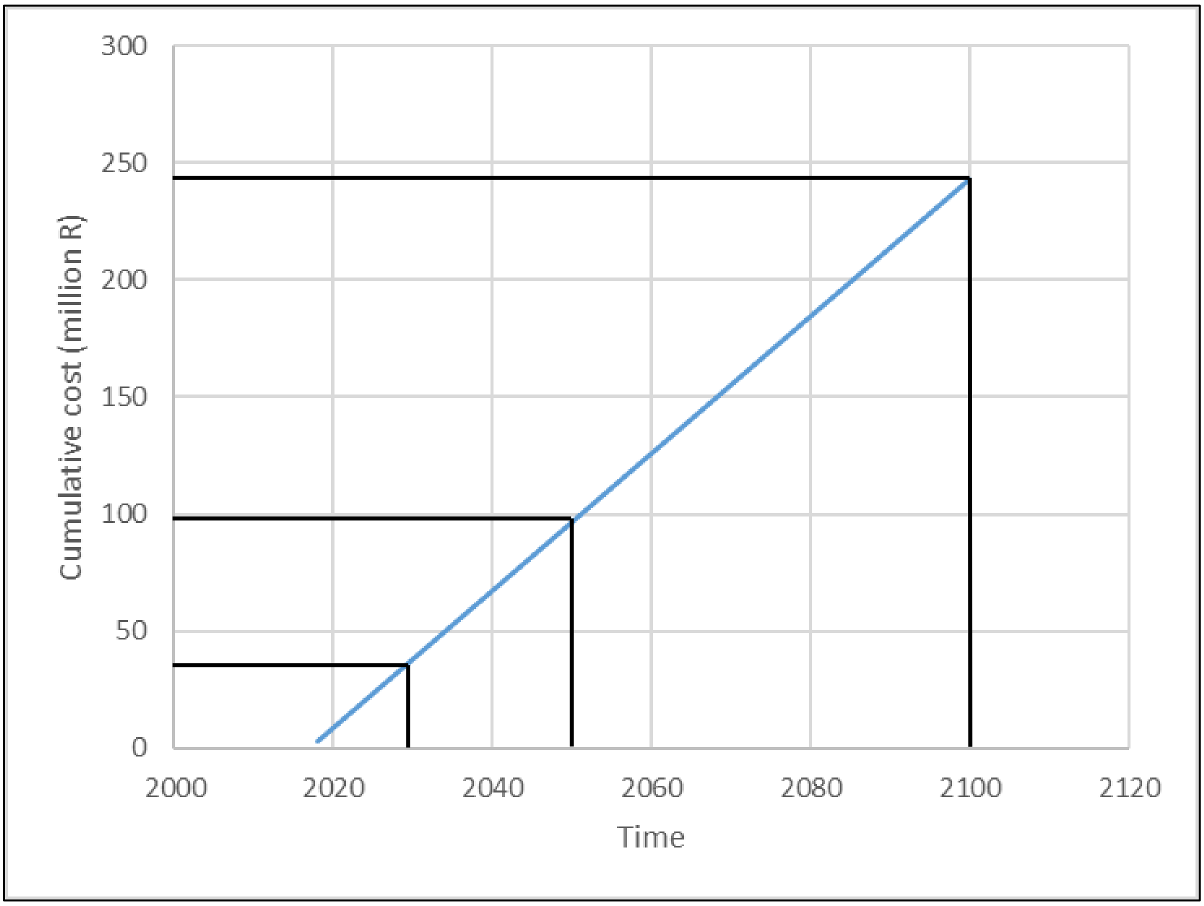


Figure 6-6: Estimated cumulative expenditure of water for emergency breaching from 2018 - 2100

value of water will increase, as the country - especially the Western Cape, becomes increasingly dryer in the next century due to climate change. This will cause the cumulative cost saved to reach the cost to build the structure far earlier. See Figure 6-6 for the estimated cumulative expenditure during the Do-Nothing alternative. At R2.93 million per year, derived from the average yearly water volume discussed in preceding paragraphs, the cumulative expenditure is estimated to reach R38 million in the year 2030, R96.5 million in 2050 and R242.7 million in the year 2100. This cumulative expenditure relationship can be used to compare to the cost of constructing a flood defence at the various design condition lifetimes.

An analysis of the project costs reported in Table 6-26 compared to the cumulative expenditure relationship for the Do-Nothing alternative can be done to obtain the year in which the hypothetical "savings" will exceed the project cost. If the project cost is exceeded by the "savings" prior to the end of structure lifetime, the project could potentially be described as economically feasible. The year in which the cumulative expenditure exceeds the project cost of the structure is reported in Table 6-27.

Table 6-27: Years in which the cumulative expenditure for water will exceed the estimated project cost for the various structures

	Year 2030				**Year 2050**				**Year 2100**			
Armoured dike:	**1:1**	**1:25**	**1:50**	**1:100**	**1:1**	**1:25**	**1:50**	**1:100**	**1:1**	**1:25**	**1:50**	**1:100**
3 m MSL	2045	2053	2054	2055	2049	2055	2056	2058	2056	2063	2065	2066
3.5 m MSL	2047	2055	2056	2057	2051	2057	2058	2059	2058	2065	2066	2068
4 m MSL	2048	2056	2057	2058	2052	2059	2060	2061	2059	2067	2068	2069
4.5 m MSL	2050	2058	2059	2060	2054	2060	2061	2063	2061	2068	2069	2071

Only the structure designed for the 1:1-year conditions in 2050, combined with a +3 m MSL crest level dike, as well as all the structures designed for the 2100 year conditions can be economically justified by the cost saved on water. From the overtopping analysis in Section 6.4, it was shown that the 1:1-year, 2050 revetment only offers adequate protection for two wave loadings, namely the 1:1-year storm wave condition in the years 2030 and 2050, which decreases its attractiveness.

The structures designed for the 2100 year conditions will essentially be "paid for" well in advance, as these structures' cost is exceeded by the cumulative cost of water between 2056 and 2071. From this analysis, it could also be concluded that a structure designed for conditions derived for the year 2075 will be economically viable and cheaper than the options for the year 2100, which could potentially make it the more attractive option.

This comparison is only a rudimentary form of assessing the financial feasibility of the proposed flood defence structure. A detailed cost/benefit analysis will be recommended to assess the Nett Present Value of the proposed solution as opposed to the Do-Nothing alternative to accurately determine which structures will be economically feasible. The cost/benefit analysis should include the cost of repairing damage to properties due to large flood events. This will likely help in the justification of the proposed solution of a structure in the lower estuary basin.

7 Conclusions and recommendations

The study investigated various potential flood defence options for the Island in the Great Brak estuary. Low-lying development in and around the estuary have experienced extensive flooding and partially driven the need to perform artificial breaches.

A situation assessment was conducted for the estuary which investigated the current Estuary Management Plan. The Wolwedans Dam, completed in 1990, severely lowered the annual run-off from the catchment area and the health state of the estuary has since been under close inspection. A Water Release Policy, based on Ecological Water Requirement studies for the watershed, has been adopted into the Estuary Management Protocol to mitigate the impact of the run-off reduction.

A literature review covered the hydrodynamics of estuaries and standard methods of quantifying the extreme flood conditions to support the derivation of applicable design conditions. Research into global and regional climate change predictions were conducted to ensure the longevity of the proposed flood alleviating measure.

7.1 Conclusions

The Extreme Still Water Level for the ocean was derived by superimposing various return period vertical residuals due to inverse barometric setup, wind effects and wave setup on the Mean High-Water Springs tidal component relevant to Mossel Bay. The extreme waves adopted for design purposes were obtained from previous studies conducted on wave recordings at the FA Platform and the Waverider Buoy in Mossel Bay itself.

The fluvial flooding component, the 100-year storm flood peak for the catchment was calculated using the Direct Run-off Hydrograph method to take into account the effect of the Wolwedans Dam. The dam proved to be a vital element in the possible flood peaks flowing into the estuary. The dam was assumed to be 100% full in the design flood estimation, and the 100-year flood peak flowing into the estuary was calculated to be 865 m^3/s. At 100% capacity, the dam attenuates the 100- year flood by 9%. A dam basin model was used to assess the impact of a less than full dam on passing flood peaks. The viability of lowering the dam level prior to a forecasted storm to increase flood attenuation was investigated through the dam basin model. It was found that in order to increase the attenuation potential of the dam by 10% to approximately 20% from the full dam condition, the dam level needs to be dropped to about 80%. A release of about 5 million m^3 of water is needed for this effect. Thus, it was concluded that flood attenuation by lowering the dam level is not feasible as less water is required to perform a full breach of the estuary mouth, which will allow the flood to pass into the ocean relatively unhindered if done early.

The wave climate at Great Brak estuary was investigated. It was deemed possible for waves to reach the Island if adverse conditions exist. If the mouth is open large enough, and an extreme Still Water Level pushes into the estuary from the sea, it will be possible for waves to propagate into the estuary area. The nearshore extreme wave condition was used to derive the depth-limited significant wave height, which was subsequently used for the concept design of the preferred flood defence measure in terms of wave penetration. The minimum representative bed level of the estuary mouth inlet was assumed to be -0.5 m MSL.

The estuary mouth berm proved to be an important hydraulic control and the variability of the mouth state imposes a large amount of uncertainty on the design conditions derived in this study, which might lead to overestimation of the extreme water levels in the lower estuary basin. The Emergency Protocol of the estuary involves the monitoring of the vital components in the watershed area that might cause flooding of low-lying properties and arranging an emergency breach of the estuary if emergency conditions prevail. The size of the berm and the water level of the ocean also play a significant role in the flushing efficiency observed during breaches. An overly large berm or a high-water level of the ocean will negatively affect the rate at which the estuary can be drained. The highest water level ever recorded in the estuary was observed to be +2.9 m MSL, and happened during a 10-year storm coinciding with a closed mouth, even though an emergency breach was performed.

It is clear from the investigation into the design conditions that larger floods, both from the catchment and from the ocean is possible, and that a flood defence measure is needed for the Island in the estuary. Various methods of flood defence were assessed in this study. Shoreline parallel structures to be applied in the surf-zone, to dissipate wave energy and to allow for the estuary mouth berm to enjoy longer open mouth conditions were identified and assessed. These structures were deemed not to be feasible, as the certainty of the desired influence is low, the foreseen negative influence on adjacent beaches and the cost does not make the option justifiable. Direct flood defence measures, applicable around the Island, like a Seawall, Revetment and Dike type structure were deemed to be more feasible and subjected to further evaluation.

A softer engineering option of a Vegetated Dune was also identified as applicable on the estuary mouth berm and evaluated further. The option was deemed to be natural and aesthetically pleasing which made it an attractive option. The dune will act as an energy dissipater of large waves and will stop the overtopping of the mouth berm. The concept was investigated in an overtopping analysis of a theoretical mouth berm feature, and proof of concept was derived from the results. The influence that the concept will have on the mouth state and the frequency of closure is uncertain. The concept will see the stabilisation of the mouth and adequate protection from large waves penetrating the estuary or overtopping the berm. However, it may exasperate fluvial flooding events if the mouth cannot be opened wide enough to allow adequate drainage of the flood and backflooding may cause extreme water levels in the estuary.

A Multi-Criteria Analysis was performed to objectively identify the preferred flood defence option. The evaluation criteria were derived to incorporate hydraulic, environmental and cost considerations. The objectives of the Estuary Management Plan were used as guidelines for the evaluation criteria. A combination of a Rubble Mound Rock Revetment and Armoured Dike structure was identified as the preferred defence options.

The study culminated in the conceptual design of the combined flood defence structure, accompanied by an order of magnitude cost estimate for the construction of the proposed solution. The Armoured Dike was designed for a crest height of +3, +3.5, +4 and +4.5 m MSL whereas the Rubble Mound Rock Revetment was designed for wave conditions derived for the 1:1-, 1:25-, 1:25- and 1:100-year storm conditions in the years 2030, 2050 and 2100 to account for SLR.

To assess the feasibility of the proposed flood defence measure, the estimated project cost for each design condition and design lifetime configuration was compared to the projected cumulative cost of the Do-Nothing alternative. The cost of water during the Do-Nothing alternative was identified as the annual operating cost and it was estimated, based on an average of R2.93 million per year, that this operating cost will cumulate to R38 million in the year 2030, R96.5 million in 2050 and R242.7 million in the year 2100 (excluding inflation or increase in water prices).

Table 7-1: Project cost (in million Rand) for all structure combinations under consideration

	Year 2030				**Year 2050**				**Year 2100**			
Armoured dike:	**1:1**	**1:25**	**1:50**	**1:100**	**1:1**	**1:25**	**1:50**	**1:100**	**1:1**	**1:25**	**1:50**	**1:100**
3 m MSL	84	107	110	113	95	114	117	120	116	137	141	145
3.5 m MSL	89	112	115	118	100	119	122	125	121	142	145	149
4 m MSL	93	117	120	123	105	124	126	130	125	147	150	154
4.5 m MSL	98	121	124	127	110	128	131	135	130	151	155	159

The result of the order of magnitude cost estimate for all the structures under consideration can be seen in Table 7-1. The structures designed for the shorter lifetimes, i.e. 2030 and 2050 can mostly not be economically justified, save for the combination of the +3.0 m MSL Armoured Dike and 1:1-year, 2050 Revetment structure, which is deemed to be economically feasible (indicated in blue). The structures designed for the year 2100 are all cheaper than the estimated cumulative cost of the Do-Nothing alternative for the same period, and can possibly be described as economically viable. However, the

Revetment structures designed for the 2100 conditions all require a crest height of over +6 m MSL, which may be too high to gain the support of the residents on the Island.

The combination solution of the +3.0 m MSL Armoured Dike (refer to Figure 6-4) and 1:1-year, 2050 Revetment structure (refer to Figure 6-3) is subsequently identified as the most attractive solution as it is the cheapest structure that can potentially be economically justified and the maximum required crest height is +4.99 m MSL, as opposed to the over +6 m MSL crest height of the structures designed for the year 2100, which will make it easier to gain the support of the residents of the Island. See Figure 6-5 for the conceptual plan layout of the abovementioned combination structure. Based on a low-level Google Earth assessment, it is concluded that there is sufficient space around the Island for the application of the proposed solution.

7.2 Recommendations

7.2.1 Structural issues

The availability of sufficient armour rock volumes and sizes for both the Armoured Dike and the Rock Revetment is an important issue to be addressed in the design phase of the project. The proximity of the source(s) of the required rock should be as close as possible to the site, as the transportation cost will likely have a significant impact on the overall cost.

Figure 7-1: Plan layout of the combination solution of the +3.0 m MSL Armoured Dike and 1:1-year, 2050 Revetment structure

The transition zones (refer to Figure 7-1) between the Armoured Dike and the Rock revetment should receive special attention in the preliminary and detailed design phases of the project as it will be a vulnerable area of the structure. It is recommended that a certain degree of overlapping of the structures should be considered, to adequately protect the core material and ultimately the structural integrity.

Storm water drainage should be adequately designed to prevent damming of water on the Island and in the lee of the proposed structure. Application of one way drainage pipes with one way flap valves should be considered.

7.2.2 Emergency evacuation protocol

The proposed solution for flood defence at the estuary will see the Island being encircled in a high crest structure to stop inundation of property. If the defence is breached, a dangerous situation will still exist for the residents of the Island, as there is currently only one entry and exit point via the bridge. It is recommended that an emergency evacuation protocol should be in place to ensure the safety of the residents.

7.2.3 Surveys

Detailed topographic and bathymetric surveys of the estuary reach and inlet area are recommended. Detailed information of the perimeter of the Island will be needed in the design phase of the proposed flood defence measures. The surveys can be utilised to support the determination of the design criteria with greater accuracy and spatial variation, in an attempt to lower the required crest height of the structure.

7.2.4 Further studies

7.2.4.1 Lowering the crest height of the proposed revetment structure

The crest height of the proposed flood defence structures that are deemed to be economically feasible, are all over +4.99 m MSL, which might impede some residents on the Islands' ocean view. This is deemed to be due to the conservative design conditions adopted in this study.

The design conditions adopted in this study can be optimised by detailed flood line mapping with the aid of hydrodynamic and morphological numerical software packages. This may help to identify the most likely depth of inlet channel, currently taken as -0.5 m MSL, which will allow waves to reach the Island during large storm events. A wave propagation numerical software package should be incorporated to assess the wave heights at the Island during a marine storm in greater detail. This can then be used to lower the crest height of the revetment for certain sections of the Island.

7.2.4.2 Detailed cost/benefit analysis

For further investigation into the feasibility of the proposed solution, a detailed cost/benefit analysis is recommended. The cost/benefit analysis can be used to accurately compare the Do-Nothing alternative with the cost implications of the flood defence structure to find the optimal solution with a balance of acceptable risk and economic feasibility.

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A. Appendix A: Estuary Monitoring

A.1 Monitoring Checklists

Table A-1: Great Brak River: Estuary Checklist (CSIR 1990)

A	CRITERIA	SCORE		SCORE?	COMMENTS
		YES	NO		
1	Is the mouth open?	2	0		Depth: m Width: m
2	Is the estuary water level less than +1.22 MSL	2	0		Level=
3	Is there a bad smell and/or excessive algal growth in the water?	0	1		If yes, please describe:
4	Is the E.Coli level less than 1000?	2	0		E.Coli level=
5	Is the salinity level more than 7 and less than 40?	2	0		Salinity level=
6	Are fish dying or under stress e.g. gaping at the surface for air?	0	2		If yes, please describe:
7	Is it February? Is it June? Is it November?	-1 -1 -2	0 0 0		
Total:					Action?
Note: OPEN MOUTH IF TOTAL <9					

Table A-2: Great Brak River: Breach monitoring (CSIR 1990)

B	Monitoring Information	Time	Date	Other
1	Water level started:			Volume released: m^3
2	Water level stopped:			
3	Mouth opening started:			Actual machine time: hrs
4	Mouth opening completed:			
5	Mouth closed on:			Was mouth re-opened?
6	Total rainfall recorded per month: mm			
7	General comments:			

A.2 Surveys

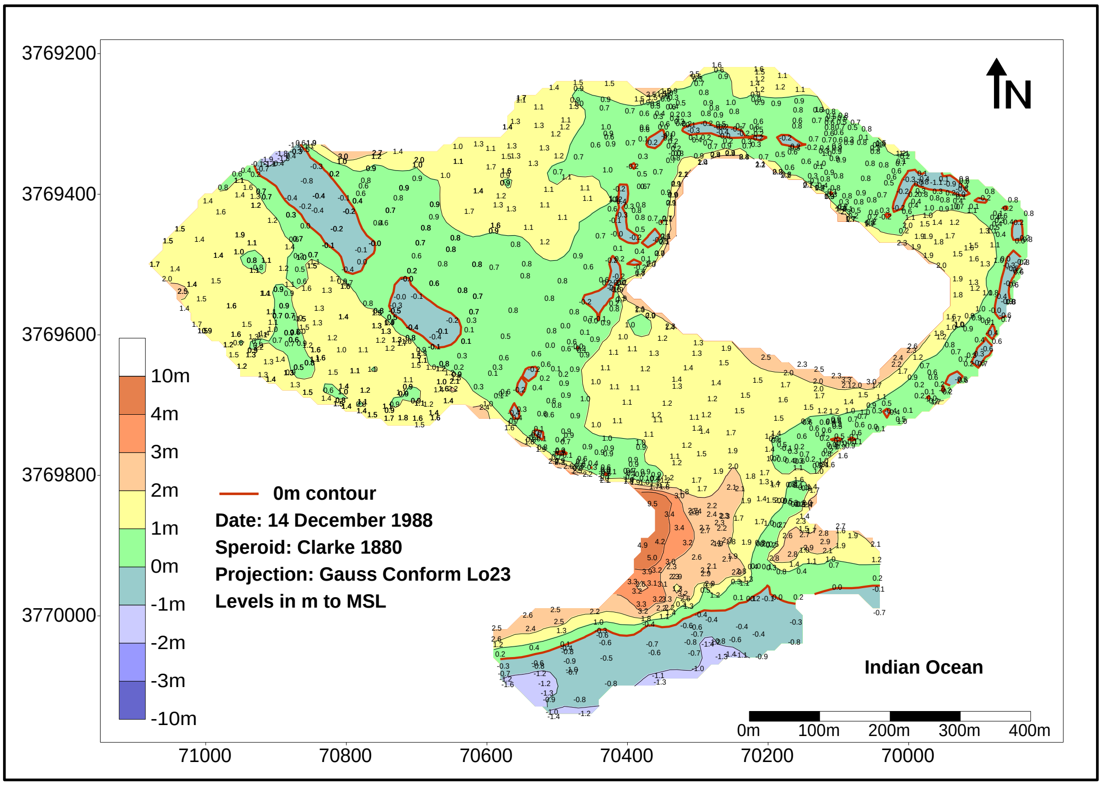


Figure A-1: Bathymetric survey of the lower estuary basin done in 1988 (Source: Huizinga 2017)

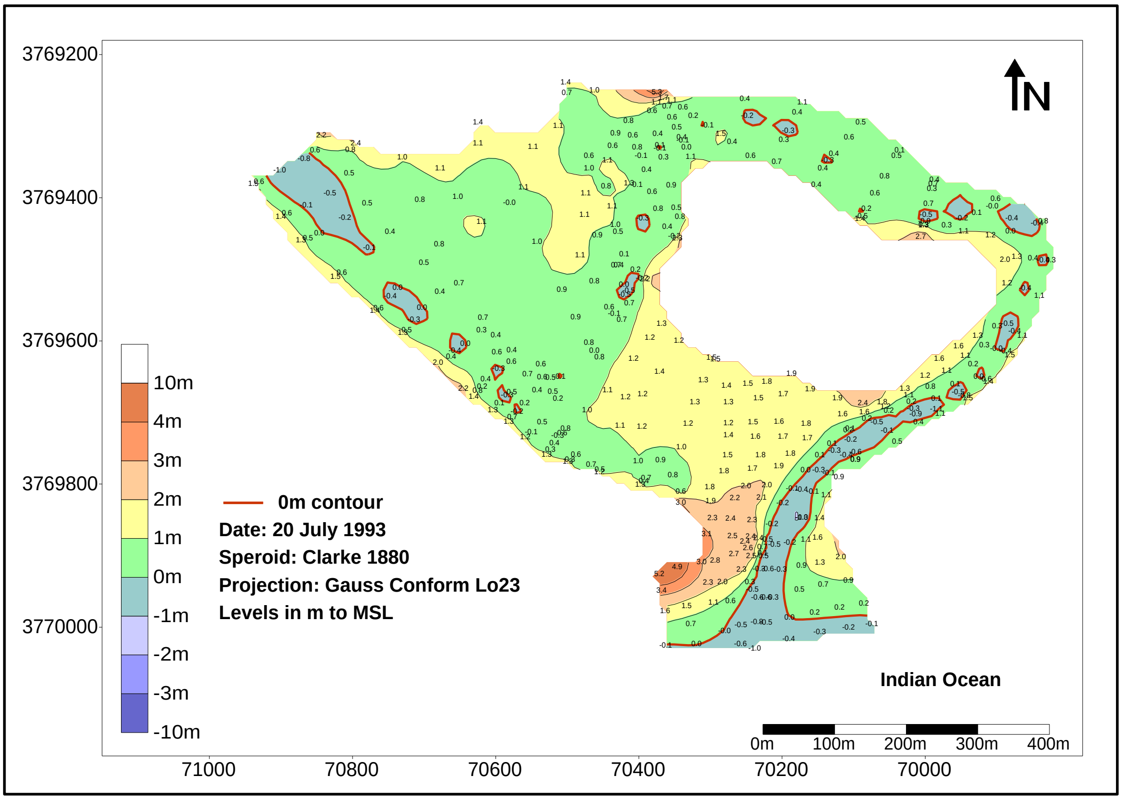


Figure A-2: Bathymetric survey of the lower estuary basin done in 1993 (Source: Huizinga 2017)

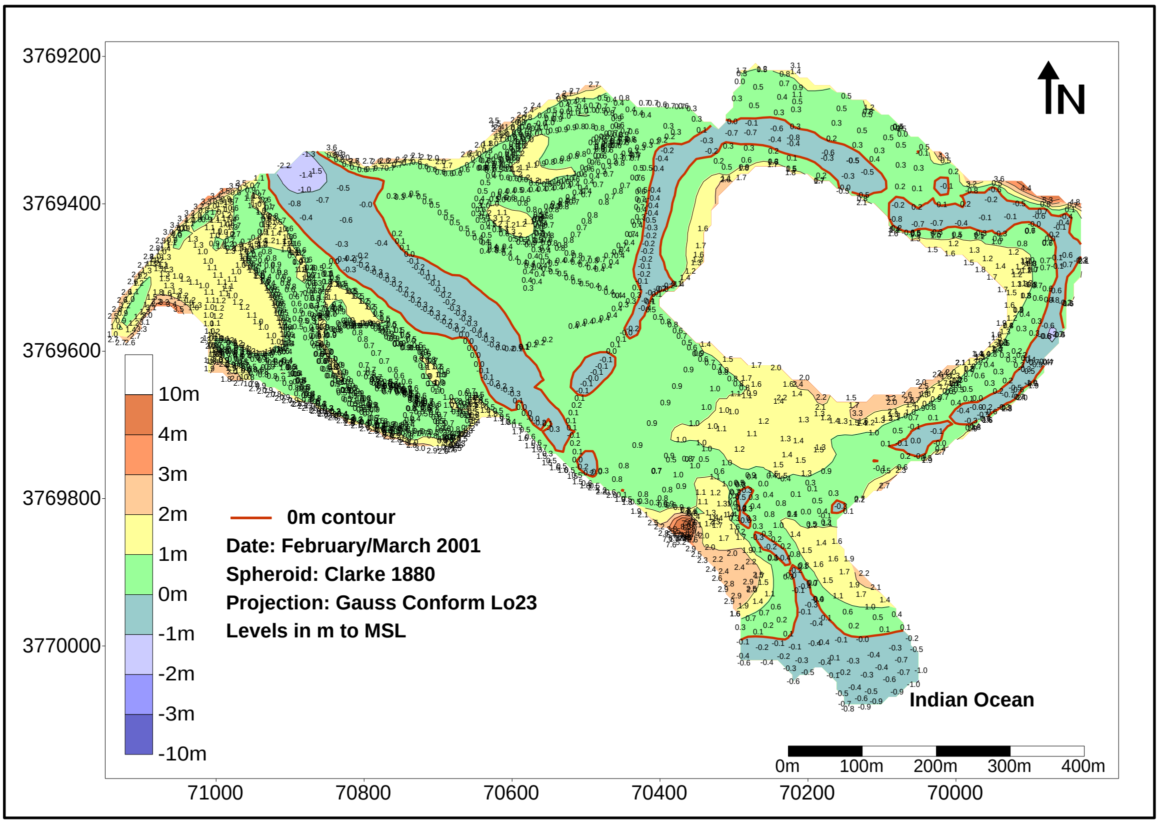


Figure A-3: Bathymetric survey of the lower estuary basin done in 2001 (Source: Huizinga 2017)

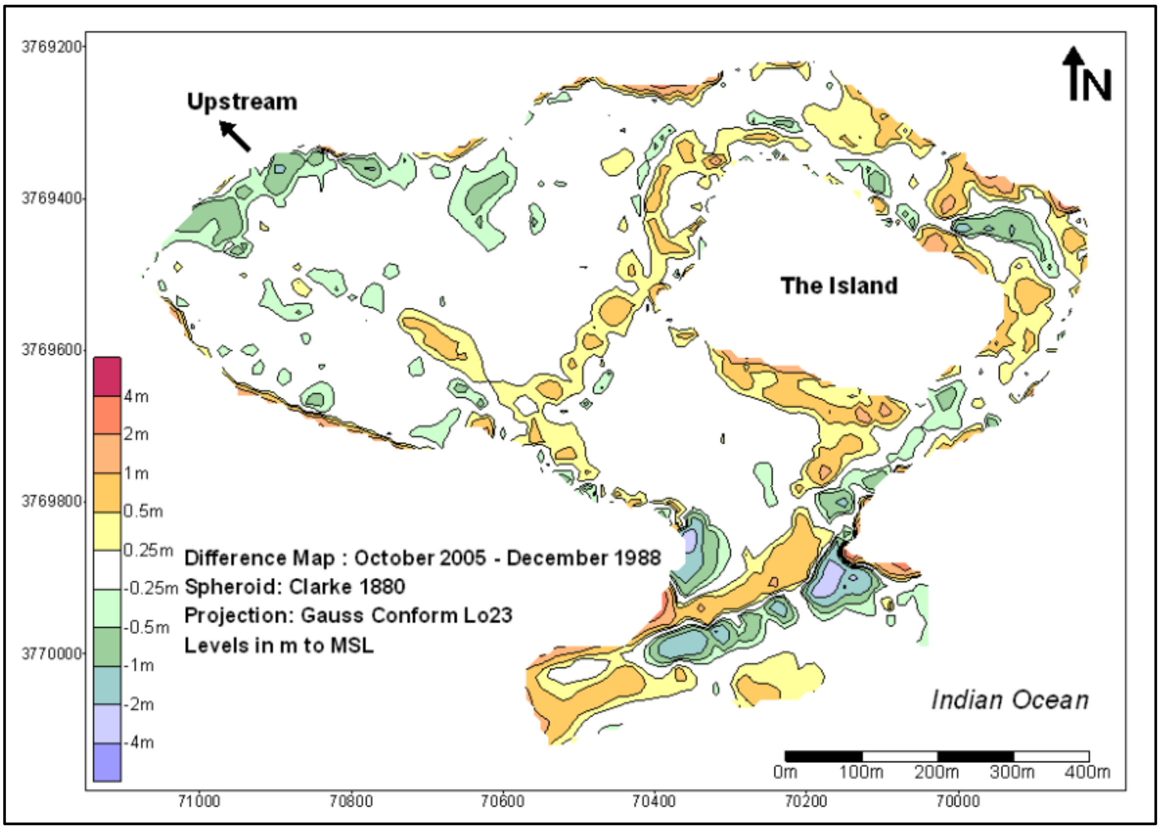


Figure A-4: Difference plot between the 1988 and 2005 survey (Source: Huizinga 2017)

A.3 Emergency protocol flow chart

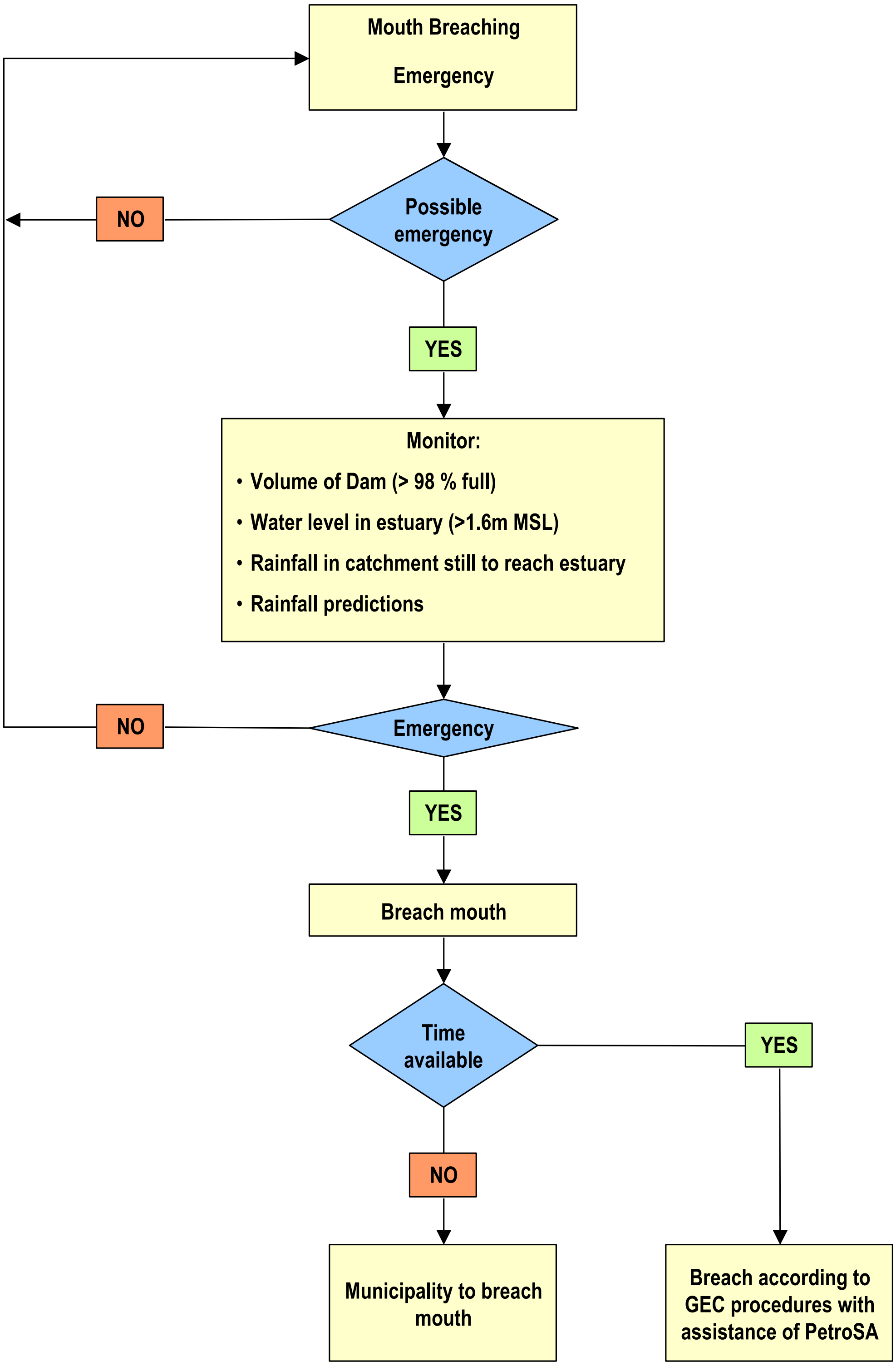


Figure A-5: Decision flow chart for the emergency breaching protocol (Source: Council for Scientific and Industrial Research 2004)

B. Appendix B: Estuary Mouth Breaching Events

Table B-1: Great Brak Estuary Mouth breaching's since 1990 (Source: Council for Scientific and Industrial Research 2011 and Jacques Kriel 2017)

DATE	WATER LEVEL BEFORE RELEASE (m TO MSL)	RELEASE VOLUME (m^3)	WATER LEVEL AFTER RELEASE (m TO MSL)	MAXIMUM OUTFLOW (m^3/s)	LOWEST WATER LEVEL AFTER BREACHING	BREACHED BY
30-11-1990	1.09	380 000	1.61	16.6	0.58	GEC
14-03-1991	1.32	230 000	1.72	17.5	0.61	GEC
09-07-1991	1.37	120 000	1.56	16.3	0.62	GEC
30-10-1991	1.44	Flood	1.77	15.8	0.57	Municipality
18-12-1991	1.39	192 000	1.68	17.5	0.65	GEC
14-02-1992	1.43	76 000	1.55	12.3	-	GEC
17-03-1992	1.48	210 000	1.78	27.8	0.48	GEC
06-05-1992	1.88	0	1.88	8.8	-	Municipality
26-06-1992	1.50	183 000	1.76	17.5	0.61	GEC
10-08-1992	1.76	0	1.76	10.8	0.55	Municipality
24-09-1992	1.32	289 000	1.75	17.5	0.60	GEC
16-10-1992	1.89	0	1.89	-	-	Natural
18-05-1993	1.78	0	1.78	6.2	-	Municipality
02-06-1993	1.90	0	1.90	20.2	0.80	GEC
19-07-1993	1.45	8 630 000	1.81	28.0	0.57	Grouting
13-09-1993	1.41	170 000	1.67	21.0	0.53	GEC
05-02-1994	-	-	1.80	-	0.49	Natural
26-05-1994	1.37	411 000	1.79	46.9	0.61	GEC
19-07-1994	1.49	400 000	1.88	29.8	0.62	GEC
02-08-1994	1.94	Small flood	1.94	> 40.0	0.42	Municipality
17-06-1995	1.94	Overflow	1.94	47.5	0.54	GEC
05-09-1995	1.42	400 000	1.86	29.8	0.40	GEC
25-08-1996	1.22	450 000	1.77	14.0	0.59	GEC
10-10-1996	1.6	300 000	1.87	31.0	0.36	GEC
21-11-1996	-	Overflow	-	320.0	0.30	Flood
26-08-1997	1.74	Overflow	2.05	70.7	0.46	GEC
04-09-1998	1.56	300 000	1.83	26.3	0.74	GEC
13-11-1998	1.44	500 000	1.99	22.2	0.62	GEC

13-03-1999	2.12	Overflow	2.12	60.3	0.53	GEC (Emergency)
24-06-1999	1.53	400 000	1.99	33.3	0.58	GEC
21-09-1999	1.50	145 000	2.02	40.0	0.52	GEC
11-11-1999	1.54	450 000	1.76	15.8	0.47	GEC
01-03-2000	1.28	500 000	1.88	26.3	0.54	GEC
22-09-2000	1.38	520 000	1.97	28.0	0.48	GEC
27-11-2000	0.94	350 000	1.39	-	0.68	Flush only
01-01-2001	1.24	Overflow	1.7	-	0.58	Natural
14-02-2001	1.48	Overflow	1.77	-	0.59	Natural+ Mun
16-04-2001	Open	Overflow	1.53	-	0.61	Natural
13-09-2001	1.64	390 000	1.98	30.1	0.56	GEC
25-05-2002	2.24	-	2.24	24.96	1.11	Natural
23-07-2002	1.60	300 000	1.92	16.64	0.91	GEC
23-08-2002	1.50	150 000	1.72	9.09	1.01	Municipality
04-09-2002	-	350 000	1.85	29.13	0.60	Flush
04-11-2002	2.15	-	2.15	20.81	0.88	Natural
17-12-2002	1.18	600 000	1.84	11.50	0.94	GEC
13-02-2003	1.14	730 000	1.99	27.05	0.68	GEC
25-03-2003	-	Flood	2.24	+100	0.55	Natural
22-08-2003	1.62	390 000	1.99	66	0.58	GEC
01-04-2004	1.00	840 000	1.96	52	0.607	GEC
06-05-2004	1.55	300 000	1.877	46	0.599	Flush
23-09-2004	1.423	630 000	2.009	39.5	0.592	GEC
12-10-2004	1.169	390 000	1.677	-	0.629	Flush
19-05-2005	1.01	200 000	1.315	13.04	0.592	Flush
29-09-2005	1.45	625 000	2.014	45	0.565	GEC
26-10-2005	1.057	200 000	1.340	14.00	0.592	Flush
2-11-2005	-	High waves	2.054	50	-	High waves
01-08-2006	-	Floods	2.245	245	0.433	Municipality
23-08-2006	-	Floods (Open mouth)	1.669	160	0.232	(Emergency)
25-09-2007	1.42	550 000	2.001	56.2	0.617	GEC
20-11-2007	1.15	150 000	1.316	-	-	Flush
22-11-2007	(open mouth)	Flood	2.194	409.5	-0.277 (29/11)	Flood
01-09-2008	1.342	High waves,	2.429	68.7		High waves

13-11-2008	1.076	overtopping berm	1.827	44.7		GEC
28-12-2008		Flood 200 000	1.52		0.643	(Preventative emergency flush)
24-03-2009	1.027	800 000	1.966	32.2	0.717	GEC
03-07-2009	1.95	0	1.95	27.5	0.718	GEC
2010						No Breaching
01-02-2011	1.35	675 000	2.02	11.44	0.969	GEC
08-06-2011	1.26	Flood	2.913	339 (infl.) 139 (infl.)	-0.166	Flood
23-06-2011	Open mouth	Flood	1.70		-0.168	Flood
14-07-2012	1.55	-	2.045	-	0.89	Emergency
03-09-2013	1.24	250 000	1.536	-	-	Flush
28-09-2013	1.75	250 000	1.99			Breach
12-06-2014	1.15	275 000*	1.561	-	-	Flush
25-06-2014	1.03	224 000*	1.406			Flush
23-09-2014	1.45	680 000	2.04			Breach
18-11-2014	1.35	300 000	1.645			Flush
17–12-2014	0.98	286 000*	1.28			Flush
14-01-2015	0.98	376 000*	1.316	-	-	Flush
01-04-2015	1.35	630 000	2.00			Breach
05-05-2015	1.22	475 500*	1.661			Flush
09-06-2015	1.78	-	1.78			Emergency
24-06-2015	1.4	278 000*	1.937			Emergency
21-07-2015	1.48	Overflow	1.92			Emergency
19-03-2016	1.03	295 000*	1.38		0.82	Flush
14-09-2016	1.63	170 000*	1.95		0.89	Emergency
13-10-2016	1.18	230 000*	1.47		0.92	Flush
10-11-2016	0.94	255 000*	1.29		0.88	Flush
28-11-2016	1.08	24 000*	1.43		0.91	Flush
09-12-2016	0.91	253 000*	1.41		1.25	Flush
27-01-2017	1.08	430 000*	1.89		-	Breach
09-03-2017	1.09	208 000*	1.32		0.83	Flush

*** - Values not reported – estimated by researcher from daily dam level history obtained from DWA (2017)**

C. Appendix C: Catchment Hydrology

The catchment hydrology was assessed in this study for extreme flood estimation for the Great Brak estuary. The DRH method was used to calculate the estimated extreme flood hydrographs. In order to take the attenuation effect of the Wolwedans Dam into consideration, the quaternary catchment area was sub-divided into the areas upstream and downstream of the dam structure. The calculations were done for both areas separately, and after flood routing was taken into account, the hydrographs for Area A and Area B were combined.

Area A

Table C-1: 100-year flood calculations for Area A utilising the DRH method

DIRECT RUNOFF HYDROGRAPH METHOD					
Description of catchment	K20A Area A - catchment area upstream of Wolwedans Dam				
River detail	Great Brak River				
Calculated by	JT Viljoen		Date	Jul-17	
Physical characteristics			Moskingum routing		
Size of Catchment (A)	123	km^2	Muskingum routing		
Longest watercourse (L)	21.57	km	factor - K	2.687	
Average slope(S_{av})	0.008238	m/m	Coefficients		
Length to catchment centroid (L_c)	10.4	km	C0	0.0792389	
MAP	730	mm	C1	0.0749382	
Veld type	2		C2	0.8458228	
Return Period (years) T =	100		Time of concentration - tc		
Rainfall				4.5	
Storm duration T_{SD}	9.00	hrs	Natural channel	$t_c = \left[\frac{0.87 \times L^2}{1000 \times S_{av}}\right]^{0.385}$	
Point rainfall, P_T	155.21	mm			
Point Intensity, P_{it}	17.25	mm/h	Flood peak		
ARF	95.98	%			
Average rainfall (P_{AvgiT})	148.97	mm	Qmax	669	m^3/s
flood run-off factor, f_{iT}	87.19	%			
Effective rain, he_{iT}	129.89	mm			

Table C-2: 50-year flood calculations for Area A utilising the DRH method

DIRECT RUNOFF HYDROGRAPH METHOD					
Description of catchment	K20A Area A - catchment area upstream of Wolwedans Dam				
River detail	Great Brak River				
Calculated by	JT Viljoen		Date	Jul-17	
Physical characteristics			Moskingum routing		
Size of Catchment (A)	123	km^2	Muskingum routing		
Longest watercourse (L)	21.57	km	factor - K	2.687	
Average slope(S_{av})	0.008238	m/m	Coefficients		
Length to catchment centroid (L_c)	10.4	km	C0	0.0788903	
MAP	730	mm	C1	0.0746279	
Veld type	2		C2	0.8464817	
Return Period (years) T =	50		Time of concentration - tc		
Rainfall				4.5	
Storm duration T_{SD}	9.00	hrs	Natural channel	$t_c = \left[\frac{0.87 \times L^2}{1000 \times S_{av}}\right]^{0.385}$	
Point rainfall, P_T	126.11	mm			
Point Intensity, P_{it}	14.01	mm/h	Flood peak		
ARF	95.98	%			
Average rainfall (P_{AvgiT})	121.04	mm	Qmax	536	m^3/s
flood run-off factor, f_{iT}	86.14	%			
Effective rain, he_{iT}	104.26	mm			

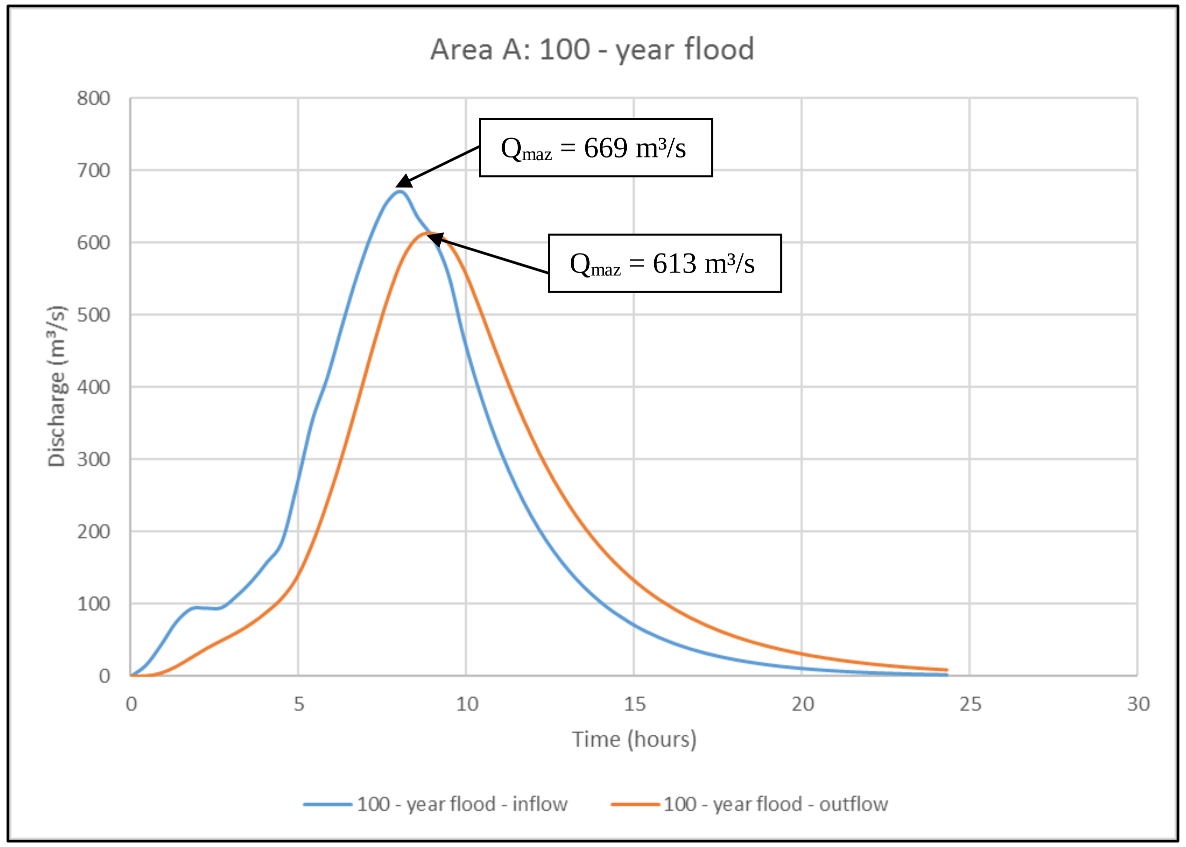


Figure C-1: 100 – year flood hydrograph Area A – unrouted and routed

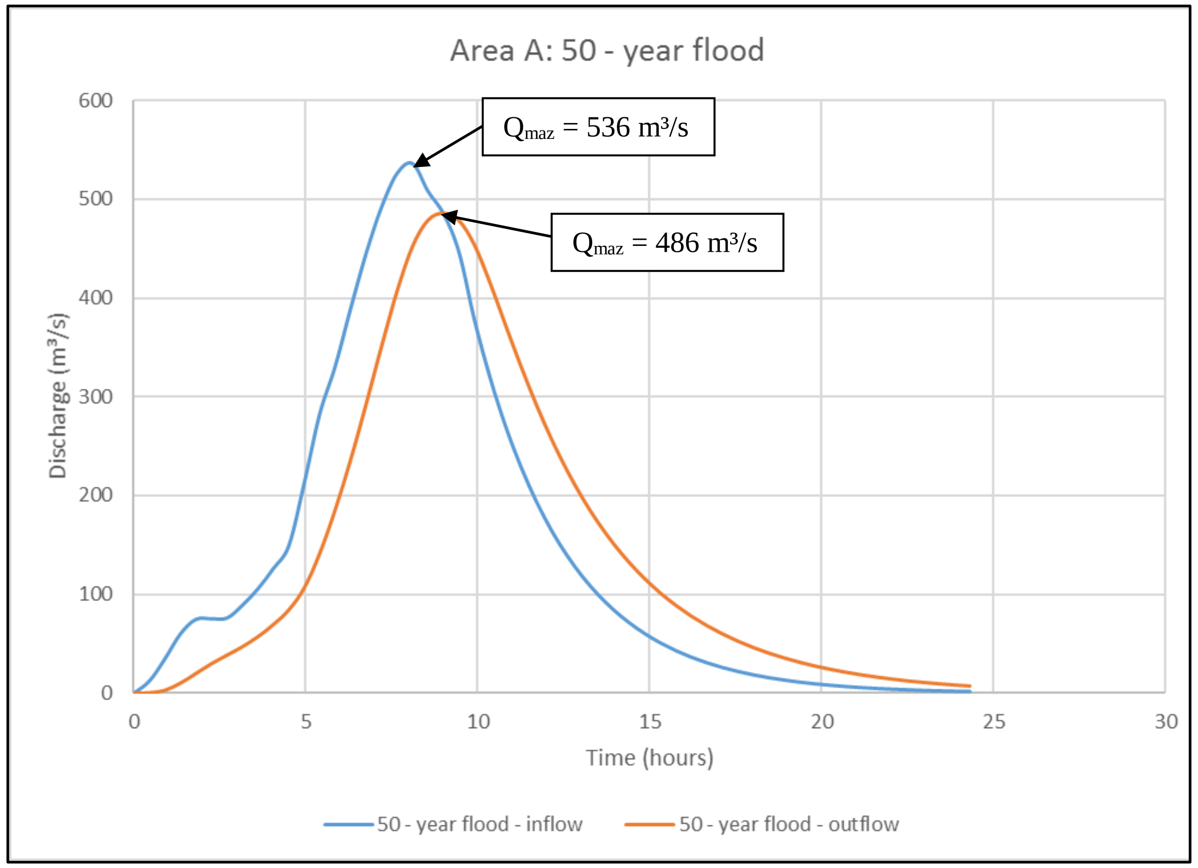


Figure C-2: 50 – year flood hydrograph for Area A – unrouted and routed

Area B

Table C-3: 100 – year flood calculations for Area B utilising the DRH method

DIRECT RUNOFF HYDROGRAPH METHOD					
Description of catchment	K20A Area B - catchment area downstream of Wolwedans Dam				
River detail	Great Brak River				
Calculated by	JT Viljoen			Date	Jul-17
Physical characteristics			Moskingum routing		
Size of Catchment (A)	41.37	km^2	Muskingum		
Longest watercourse (L)	8.027	km	routing factor - K	1.320	
Average slope(S_{av})	0.007699	m/m	Coefficients		
Length to catchment centroid (L_c)	5.91	km	C0	0.152629	
MAP	730	mm	C1	0.136247	
Veld type	2		C2	0.711124	
Return Period (years) T =	100		Time of concentration - tc		
Rainfall				2.2	
Storm duration T_{SD}	9.00	hrs	Natural channel	$c = \left[\frac{0.87 \times L^2}{1000 \times S_{av}}\right]^{0.385}$	
Point rainfall, P_T	155.21	mm			
Point Intensity, P_{it}	17.25	mm/h	Flood peak		
ARF	101.66	%			
Average rainfall (P_{AvgiT})	157.79	mm	Qmax	295	m^3/s
flood run-off factor, f_{iT}	87.48	%			
Effective rain, he_{iT}	**138.03**	mm			

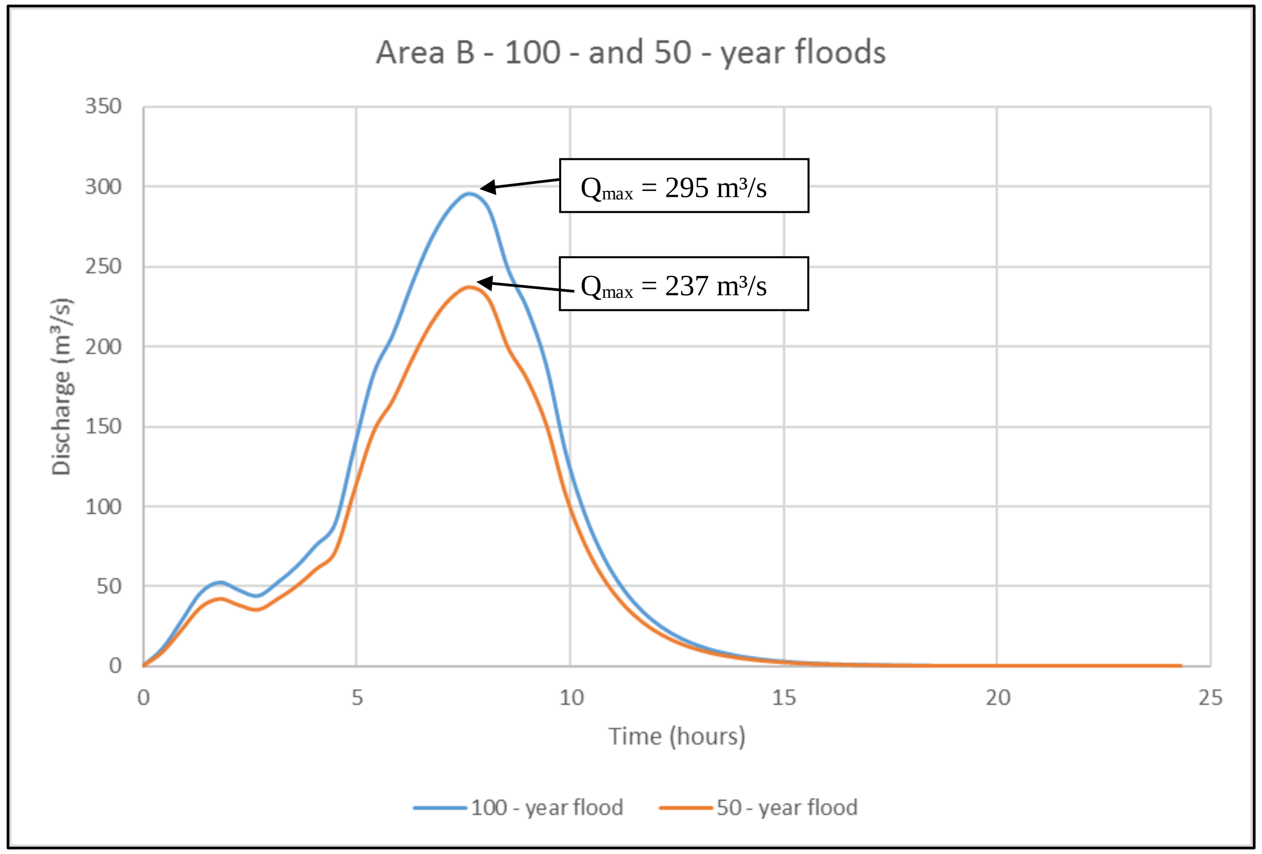


Figure C-3: 50- and 100-year flood hydrographs for Area B

Table C-4: 50 – year flood calculations for Area B utilising the DRH method

DIRECT RUNOFF HYDROGRAPH METHOD					
Description of catchment	K20A Area B - catchment area downstream of Wolwedans Dam				
River detail	Great Brak River				
Calculated by	JT Viljoen			Date	Jul-17
Physical characteristics			Moskingum routing		
Size of Catchment (A)	41.37	km^2	Muskingum		
Longest watercourse (L)	8.027	km	routing factor - K	1.320	
Average slope(S_{av})	0.007699	m/m	Coefficients		
Length to catchment centroid (L_c)	5.91	km	C0	0.152629	
MAP	778	mm	C1	0.136247	
Veld type	2		C2	0.711124	
Return Period (years) T =	50		Time of concentration - tc		
Rainfall				2.2	
Storm duration T_{SD}	9.00	hrs	Natural channel	$c = \left[\frac{0.87 \times L^2}{1000 \times S_{av}}\right]^{0.385}$	
Point rainfall, P_T	126.11	mm			
Point Intensity, P_{it}	14.01	mm/h	Flood peak		
ARF	101.66	%			
Average rainfall (P_{AvgiT})	128.20	mm	Qmax	237	m^3/s
flood run-off factor, f_{iT}	86.43	%			
Effective rain, he_{iT}	**110.80**	mm			

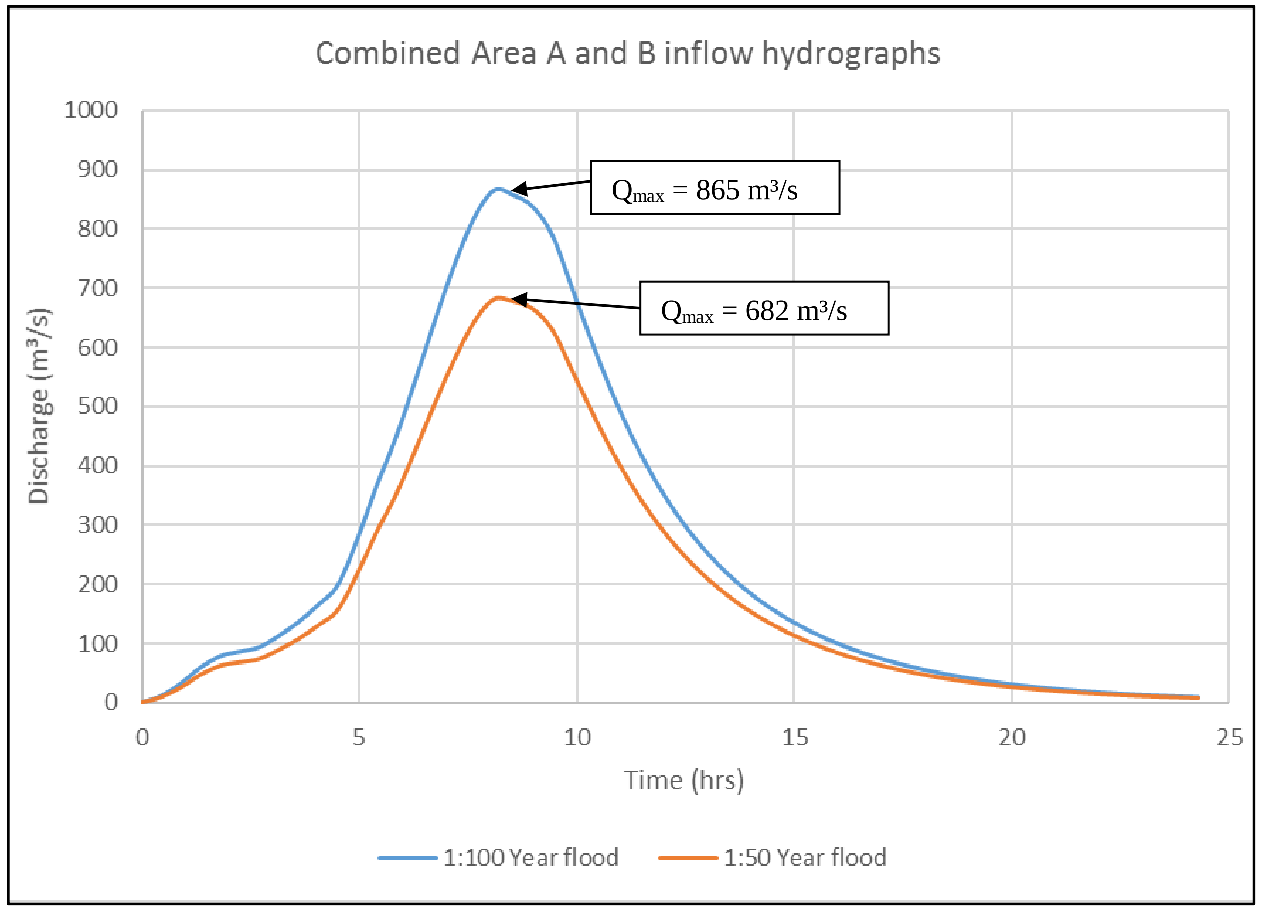


Figure C-4: Combined Area A and Area B flood hydrographs

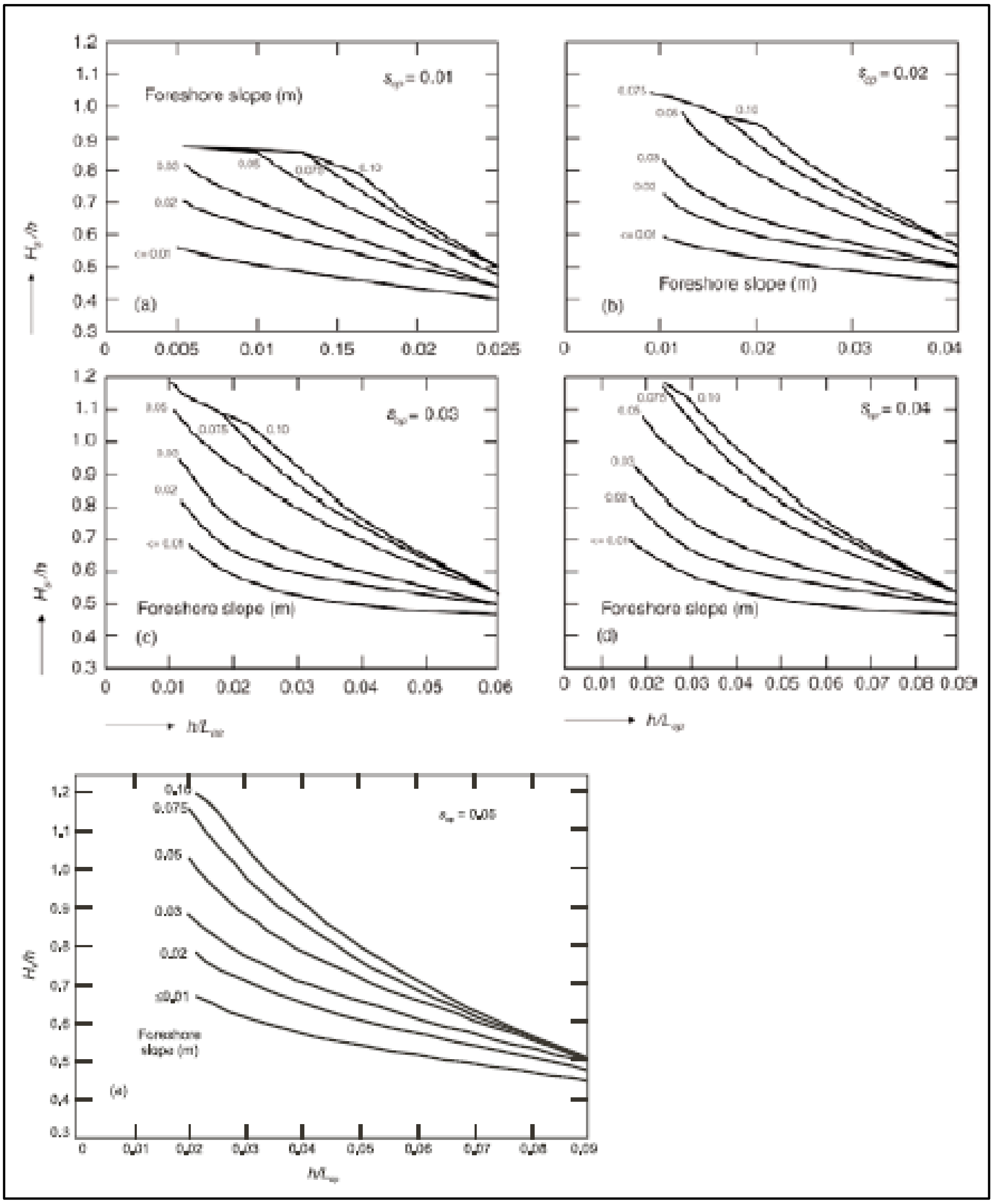


Figure C-5: Van der Meer (1990) shallow water wave heights on uniform sloping foreshore (Source: CIRIA 2007)

D.Appendix D: Flood Routing and Dam Basin Model

The Wolwedans Dam is an important hydraulic control structure 8 km upstream of the Great Brak estuary. The dam has a 123 km² catchment area, equal to three quarters of the total quaternary catchment size. The flood attenuation that the dam imposes on passing extreme floods can be approximated by the level-pool routing technique, described in Section 2.6.2. The inflow hydrographs used for routing purposes were generated by the Direct Run-off Hydrograph method, the input parameters and calculations are discussed in Appendix C. From information obtained from the Department of Water Affairs, the dam specific characteristics, like the stage discharge curve and the water level volume relationship, were derived. See Figure D-1 for the water level – volume relationship for the Wolwedans dam and Figure D-2 for the stage discharge curve.

The routing calculations were initially done with the assumption of a dam at 100%. With these dam characteristics, the estimated extreme inflow hydrographs and the assumption of an initial dam level, Equations 2-13 to 2-15 were used to develop a dam basin model, always satisfying the continuity principle. The integrals of the inflow and outflow hydrographs were calculated for a chosen time interval, *dt*, and the difference was the change in volume over that time step.

The basin model was subsequently altered to incorporate a variable initial water level, to assess what the effect of a less than full dam will have on a coincidental flood. To validate the dam basin model, two actual flood hydrographs were used, obtained from Roux and Rademeyer (2012). The June 2011

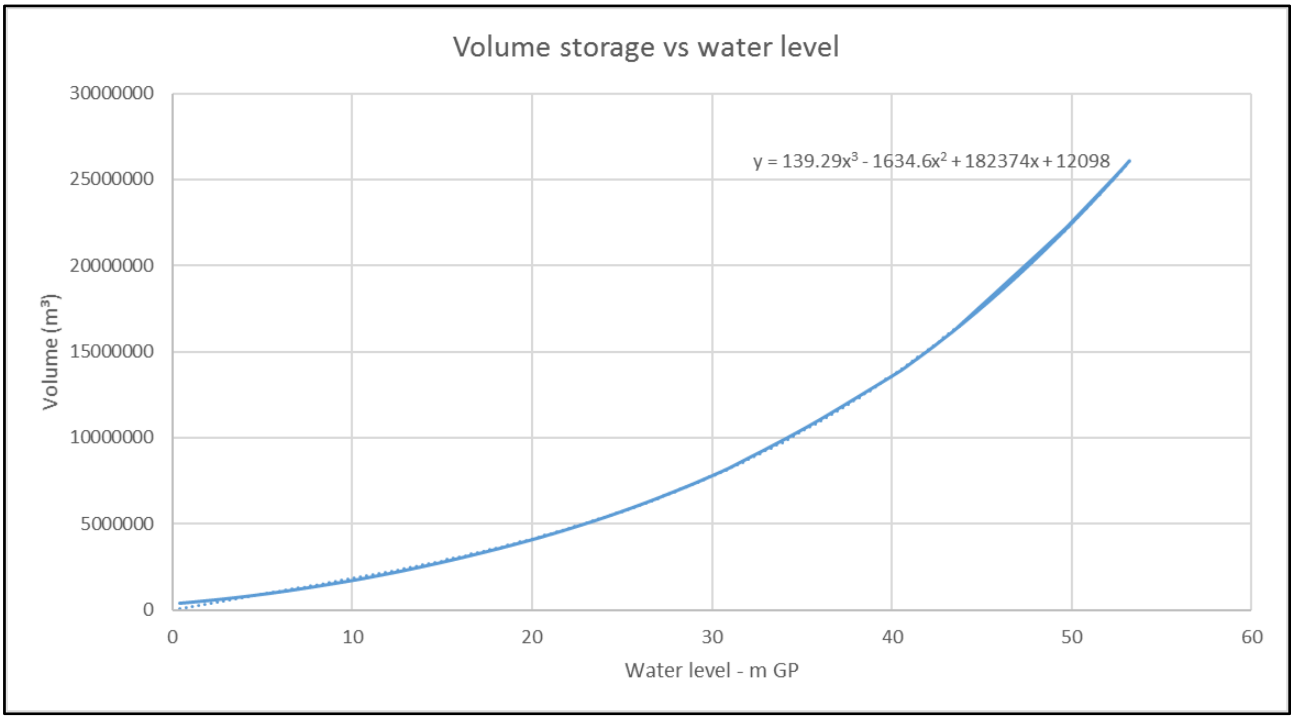


Figure D-1: Volume of storage vs water level of the Wolwedans Dam

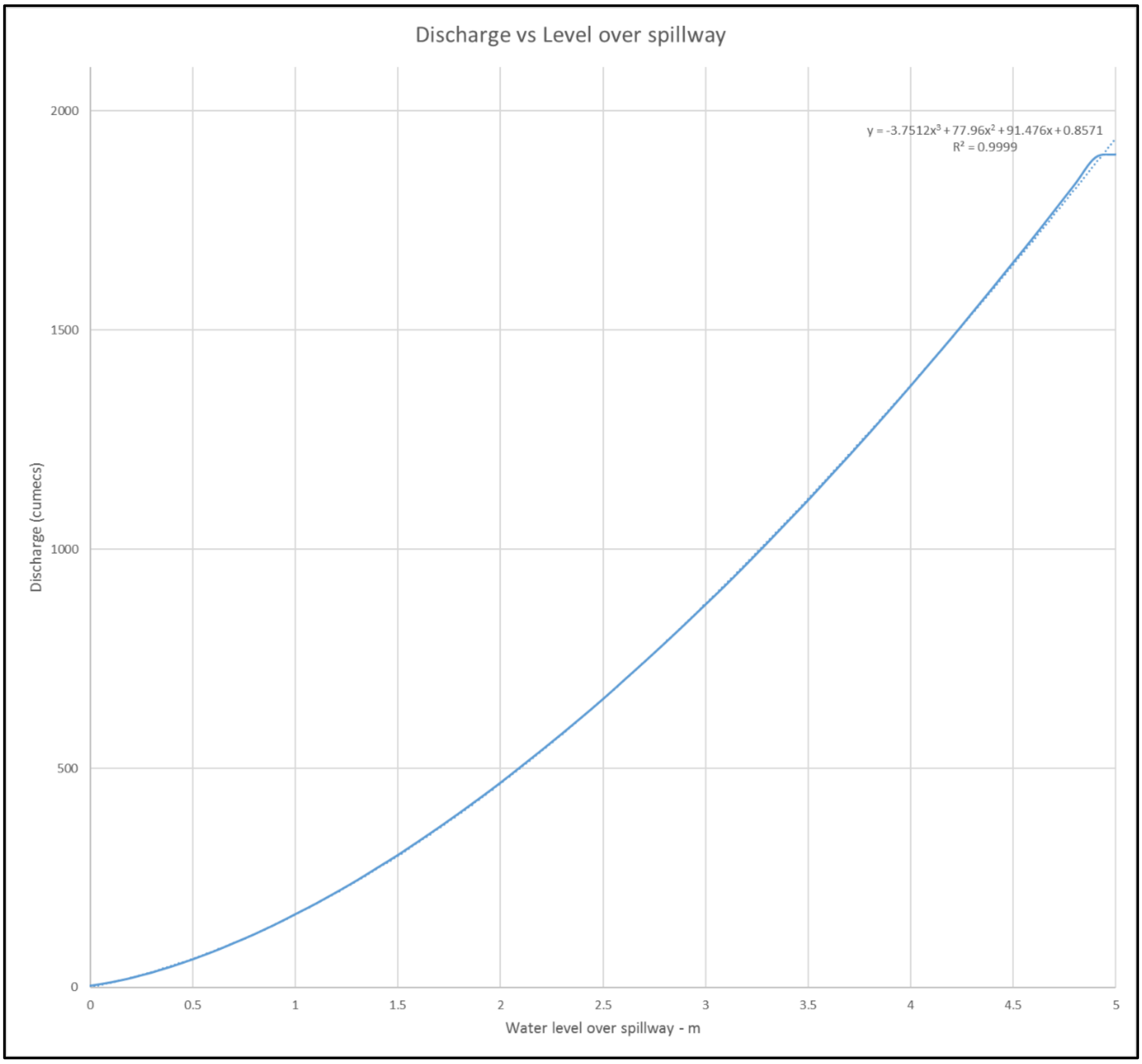


Figure D-2: Stage discharge curve for the Wolwedans Dam

and November 2007 floods were used to validate the dam basin model. The flood hydrographs were recorded at the Wolwedans Dam and coincided with less than full dam conditions, which caused significant attenuation to the peak outflow. The dam level was at 88% before the June 2011 flood and at 65% before the November 2007 flood and were attenuated 39% and 35.5% respectively. From the dam basin model, for the same initial dam level, the attenuation was calculated to be 36% (2007) and 37% (2011), in good agreement with the recorded flood hydrographs and water levels. See Figure D-3 and Figure D-5 for the recorded inflow flood hydrographs and the calculated outflow hydrographs. Figure D-4 and Figure D-6 shows the recorded flood hydrographs

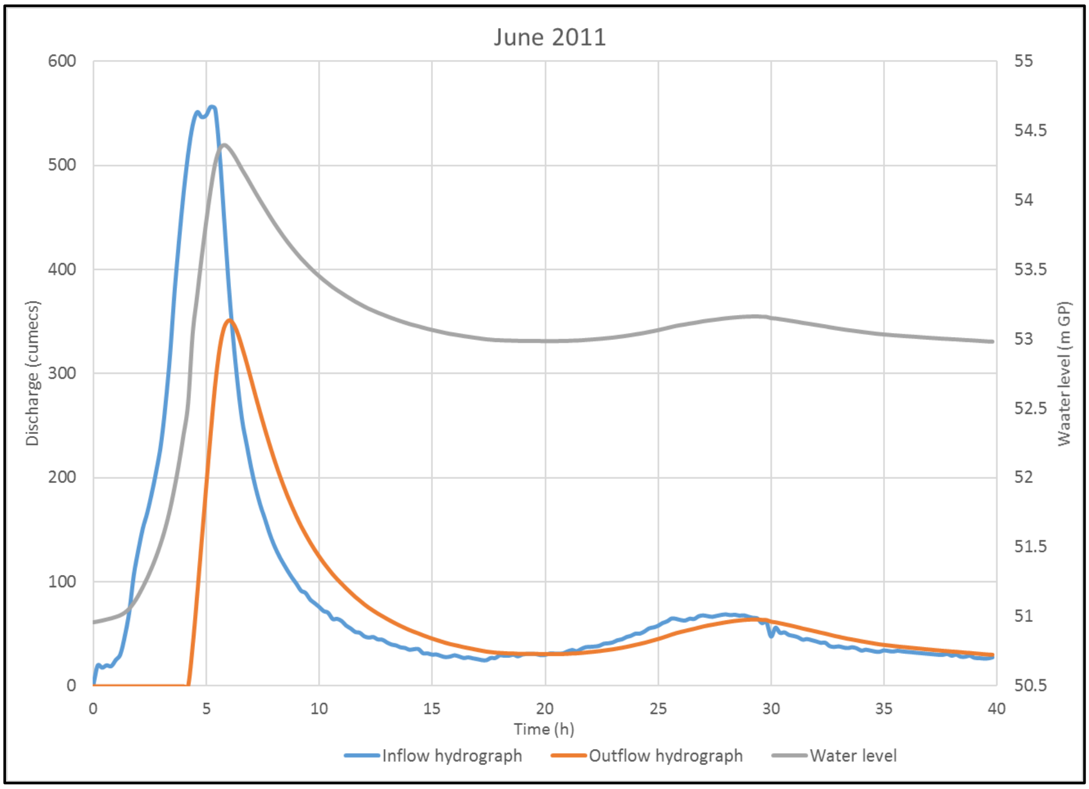


Figure D-3: Recorded inflow and calculated water level and outflow hydrographs for the June 2011 flood

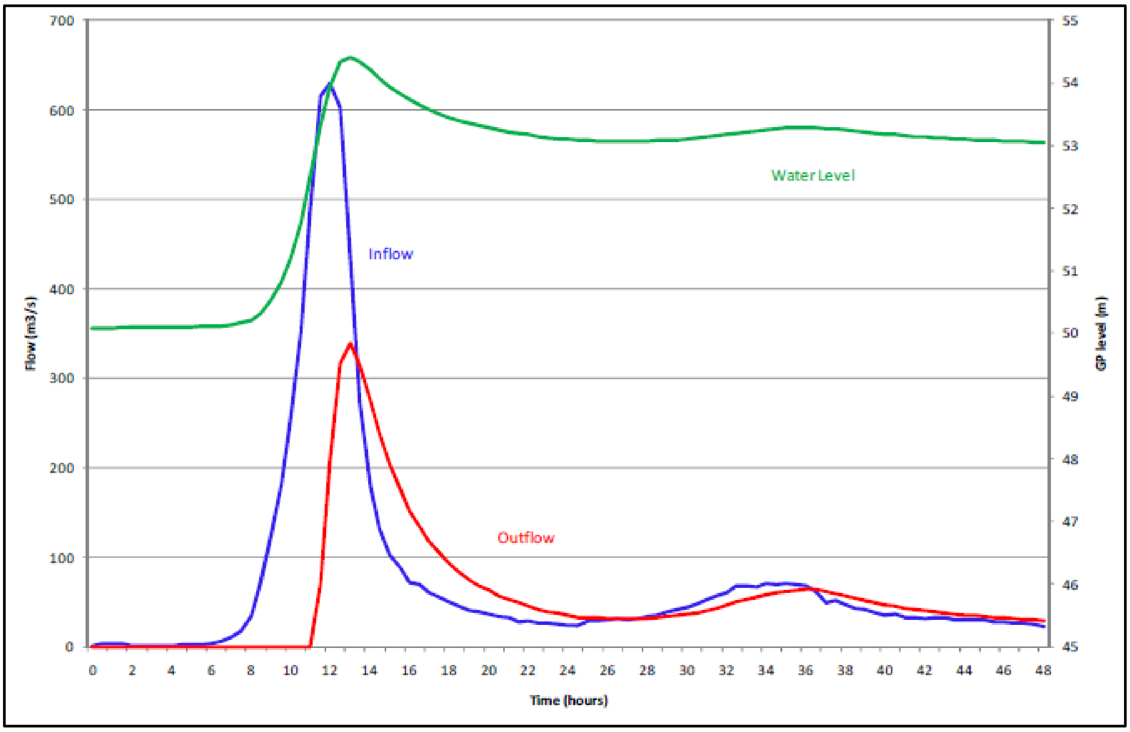


Figure D-4: Recorded water level, inflow and outflow hydrographs for the June 2011 flood (CSIR 2011)

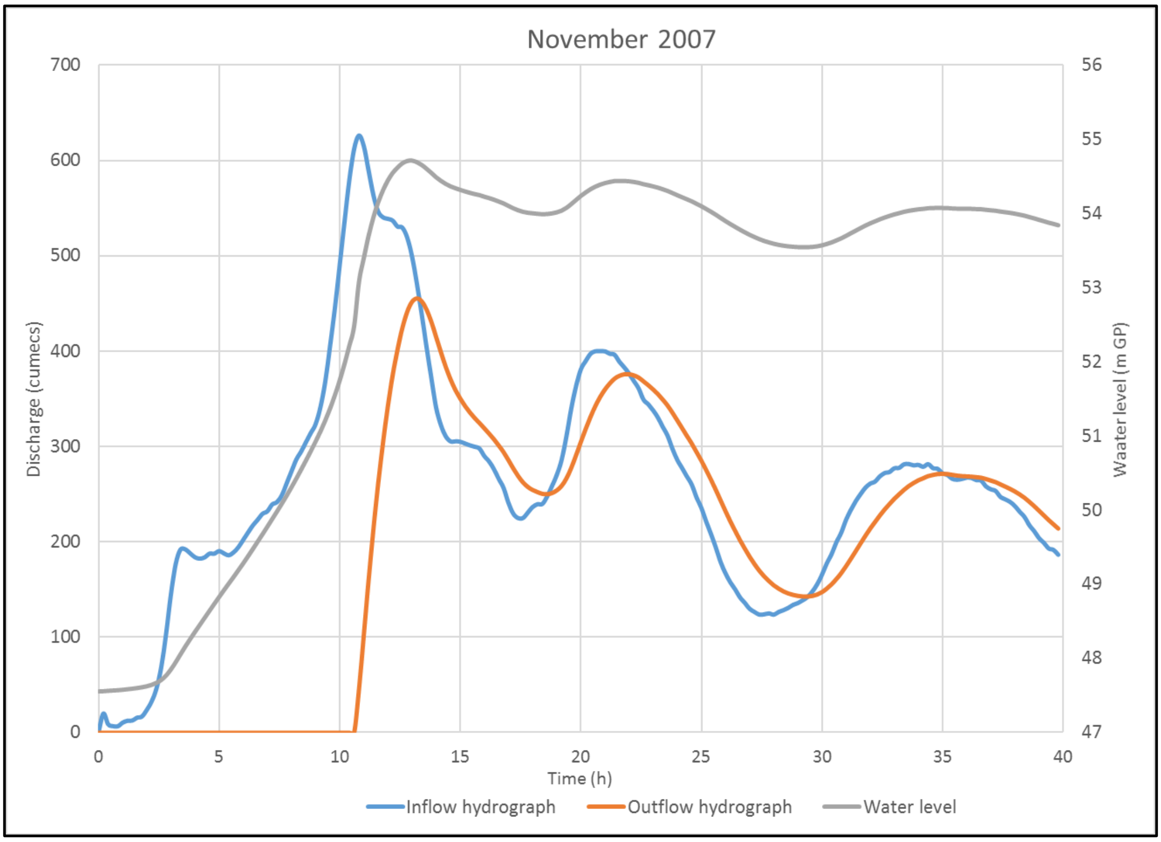


Figure D-5: Recorded inflow and calculated outflow hydrographs for the November 2007 flood

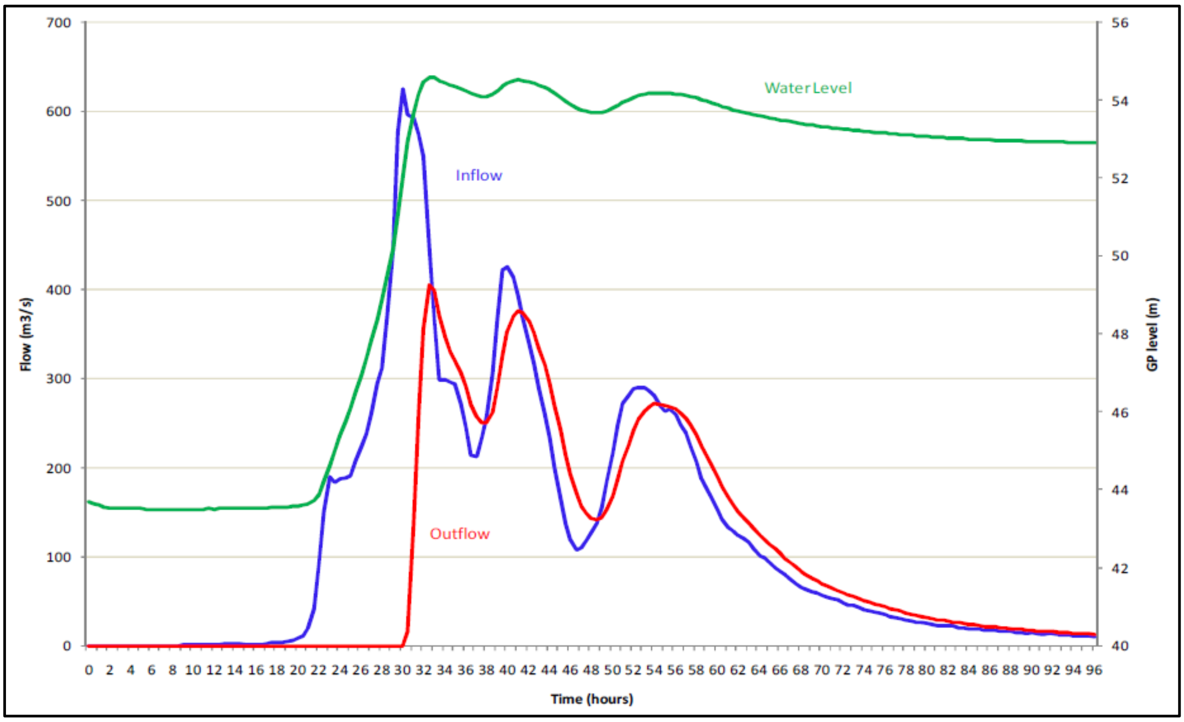


Figure D-6: Recorded water level, inflow and outflow hydrographs for the June 2011 flood (CSIR 2011)

E. Appendix E: Design of Flood Defence Measures

The preferred solution for flood defence was identified in this study, by a MCA, to be a combination of a rubble mound revetment and an armoured dike structure directly around the Island perimeter. The design considerations pertaining the development of concept cross-sectional drawings and plan layout will be discussed in this appendix.

E.1 Design of the Rubble mound revetment

E.1.1 Armour stone

Table E-1: Armour stone requirement for the 1-year incident wave and year 2030 SWL

Parameter	Slope angle: 1:3	Slope angle: 1:2	Slope angle: 1:1.5
Tanα	0.333	0.5	0.6667
Depth at toe	2.37	2.37	2.37
H_S	1.53	1.53	1.53
T_P	11.9	11.9	11.9
ξ_{cr}	2.41	3.38	4.30
$\xi_{s-1,0}$	3.64	5.46	7.28
$H_S/H_{2\%}$	0.8	0.8	0.8
S_d	2	2	2
N – 2-hour storm	665.5	665.5	665.5
Permeable – P	0.4	0.4	0.4
Required Armour stone Dn_{50} - m	0.44	0.46	0.47
Required Armour stone Mn_{50} - kg	225.51	254.67	277.63
Impermeable – P	0.1	0.1	0.1
Required Armour stone Dn_{50}	0.54	0.64	0.71
Required Armour stone Mn_{50}	420.12	683.41	965.18

Table E-2: Armour stone requirement for the 25-year incident wave and year 2030 SWL

Parameter	Slope angle: 1:3	Slope angle: 1:2	Slope angle: 1:1.5
Tanα	0.333	0.5	0.6667
Depth at toe	2.70	2.70	2.70
H_S - m	1.82	1.82	1.82
T_P - s	13.1	13.1	13.1
ξ_{cr}	2.41	3.38	4.30
$\xi_{s-1,0}$	3.68	5.51	7.35
$H_S/H_{2\%}$	0.8	0.8	0.8
S_d	2	2	2
N – 2-hour storm	604.6	604.6	604.6
Permeable – P	0.4	0.4	0.4
Required Armour stone D_{n50} - m	0.52	0.54	0.55
Required Armour stone M_{n50} - kg	364.07	411.16	448.22
Impermeable – P	0.1	0.1	0.1
Required Armour stone D_{n50} - m	0.64	0.75	0.84
Required Armour stone M_{n50} - kg	684.11	1112.85	1571.68

Table E-3: Armour stone requirement for the 50-year incident wave and year 2030 SWL

Parameter	Slope angle: 1:3	Slope angle: 1:2	Slope angle: 1:1.5
Tanα	0.333	0.5	0.6667
Depth at toe	2.74	2.74	2.74
H_S - m	1.87	1.87	1.87
T_P - s	13.3	13.3	13.3
ξ_{cr}	2.41	3.38	4.30
$\xi_{s-1,0}$	3.69	5.53	7.37
$H_S/H_{2\%}$	0.8	0.8	0.8
S_d	2	2	2
N – 2-hour storm	595.5	595.5	595.5
Permeable – P	0.4	0.4	0.4
Required Armour stone D_{n50} - m	0.53	0.55	0.57
Required Armour stone M_{n50} - kg	388.45	438.69	478.23
Impermeable – P	0.1	0.1	0.1
Required Armour stone D_{n50} - m	0.65	0.77	0.86
Required Armour stone M_{n50} - kg	731.89	1190.57	1681.44

Table E-4: Armour stone requirement for the 100-year incident wave and year 2030 SWL

Parameter	Slope angle: 1:3	Slope angle: 1:2	Slope angle: 1:1.5
Tanα	0.333	0.5	0.6667
Depth at toe	2.78	2.78	2.78
H_S - m	1.91	1.91	1.91
T_P - s	13.6	13.6	13.6
ξ_{cr}	2.41	3.38	4.30
$\xi_{s-1,0}$	3.72	5.58	7.44
$H_S/H_{2\%}$	0.8	0.8	0.8
S_d	2	2	2
N – 2-hour storm	582.4	582.4	582.4
Permeable – P	0.4	0.4	0.4
Required Armour stone D_{n50} - m	0.54	0.56	0.58
Required Armour stone M_{n50} - kg	411.97	465.26	507.20
Impermeable – P	0.1	0.1	0.1
Required Armour stone D_{n50} - m	0.67	0.78	0.88
Required Armour stone M_{n50} - kg	782.88	1273.51	1798.58

Table E-5: Armour stone requirement for the 1-year incident wave and year 2050 SWL

Parameter	Slope angle: 1:3	Slope angle: 1:2	Slope angle: 1:1.5
Tanα	0.333	0.5	0.6667
Depth at toe	2.57	2.57	2.57
H_S - m	1.62	1.62	1.62
T_P - s	11.9	11.9	11.9
ξ_{cr}	2.41	3.38	4.30
$\xi_{s-1,0}$	3.54	5.30	7.07
$H_S/H_{2\%}$	0.8	0.8	0.8
S_d	2	2	2
N – 2-hour storm	665.5	665.5	665.5
Permeable – P	0.4	0.4	0.4
Required Armour stone D_{n50} - m	0.47	0.49	0.51
Required Armour stone M_{n50} - kg	278.04	314.00	342.30
Impermeable – P	0.1	0.1	0.1
Required Armour stone D_{n50} - m	0.58	0.68	0.76
Required Armour stone M_{n50} - kg	504.60	820.84	1159.28

Table E-6: Armour stone requirement for the 25-year incident wave and year 2050 SWL

Parameter	Slope angle: 1:3	Slope angle: 1:2	Slope angle: 1:1.5
Tanα	0.333	0.5	0.6667
Depth at toe	2.90	2.90	2.90
H_S - m	1.91	1.91	1.91
T_P - s	13.1	13.1	13.1
ξ_{cr}	2.41	3.38	4.30
$\xi_{s-1,0}$	3.59	5.38	7.17
$H_S/H_{2\%}$	0.8	0.8	0.8
S_d	2	2	2
N – 2-hour storm	604.6	604.6	604.6
Permeable – P	0.4	0.4	0.4
Required Armour stone Dn_{50} - m	0.55	0.57	0.59
Required Armour stone Mn_{50} - kg	434.53	490.74	534.97
Impermeable – P	0.1	0.1	0.1
Required Armour stone Dn_{50} - m	0.67	0.79	0.88
Required Armour stone Mn_{50} - kg	798.66	1299.18	1834.83

Table E-7: Armour stone requirement for the 50-year incident wave and year 2050 SWL

Parameter	Slope angle: 1:3	Slope angle: 1:2	Slope angle: 1:1.5
Tanα	0.333	0.5	0.6667
Depth at toe	2.94	2.94	2.94
H_S - m	1.96	1.96	1.96
T_P - s	13.3	13.3	13.3
ξ_{cr}	2.41	3.38	4.30
$\xi_{s-1,0}$	3.60	5.40	7.20
$H_S/H_{2\%}$	0.8	0.8	0.8
S_d	2	2	2
N – 2-hour storm	595.5	595.5	595.5
Permeable – P	0.4	0.4	0.4
Required Armour stone Dn_{50} - m	0.56	0.58	0.60
Required Armour stone Mn_{50} - kg	461.71	521.43	568.43
Impermeable – P	0.1	0.1	0.1
Required Armour stone Dn_{50} - m	0.68	0.81	0.90
Required Armour stone Mn_{50} - kg	851.34	1384.88	1955.86

Table E-8: Armour stone requirement for the 100-year incident wave and year 2050 SWL

Parameter	Slope angle: 1:3	Slope angle: 1:2	Slope angle: 1:1.5
Tanα	0.333	0.5	0.6667
Depth at toe	2.98	2.98	2.98
H_S - m	2.01	2.01	2.01
T_P - s	13.6	13.6	13.6
ξ_{cr}	2.41	3.38	4.30
$\xi_{s-1,0}$	3.64	5.45	7.27
$H_S/H_{2\%}$	0.8	0.8	0.8
S_d	2	2	2
N – 2-hour storm	582.4	582.4	582.4
Permeable – P	0.4	0.4	0.4
Required Armour stone D_{n50} - m	0.57	0.59	0.61
Required Armour stone M_{n50} - kg	487.59	550.66	600.29
Impermeable – P	0.1	0.1	0.1
Required Armour stone D_{n50} - m	0.70	0.82	0.92
Required Armour stone M_{n50} - kg	907.25	1475.83	2084.31

Table E-9: Armour stone requirement for the 1-year incident wave and year 2100 SWL

Parameter	Slope angle: 1:3	Slope angle: 1:2	Slope angle: 1:1.5
Tanα	0.333	0.5	0.6667
Depth at toe	3.22	3.22	3.22
H_S - m	1.92	1.92	1.92
T_P - s	11.9	11.9	11.9
ξ_{cr}	2.41	3.38	4.30
$\xi_{s-1,0}$	3.25	4.88	6.50
$H_S/H_{2\%}$	0.8	0.8	0.8
S_d	2	2	2
N – 2-hour storm	665.5	665.5	665.5
Permeable – P	0.4	0.4	0.4
Required Armour stone D_{n50} - m	0.58	0.60	0.62
Required Armour stone M_{n50} - kg	510.16	576.15	628.09
Impermeable – P	0.1	0.1	0.1
Required Armour stone D_{n50} - m	0.69	0.81	0.91
Required Armour stone M_{n50} - kg	858.24	1396.11	1971.72

Table E-10: Armour stone requirement for the 25-year incident wave and year 2100 SWL

Parameter	Slope angle: 1:3	Slope angle: 1:2	Slope angle: 1:1.5
Tanα	0.333	0.5	0.6667
Depth at toe	3.55	3.55	3.55
H_S - m	2.21	2.21	2.21
T_P - s	13.1	13.1	13.1
ξ_{cr}	2.41	3.38	4.30
$\xi_{s-1,0}$	3.34	5.00	6.67
$H_S/H_{2\%}$	0.8	0.8	0.8
S_d	2	2	2
N – 2-hour storm	604.6	604.6	604.6
Permeable – P	0.4	0.4	0.4
Required Armour stone Dn_{50} - m	0.65	0.68	0.70
Required Armour stone Mn_{50} - kg	731.96	826.64	901.15
Impermeable – P	0.1	0.1	0.1
Required Armour stone Dn_{50} - m	0.78	0.92	1.03
Required Armour stone Mn_{50} - kg	1260.43	2050.35	2895.71

Table E-11: Armour stone requirement for the 50-year incident wave and year 2100 SWL

Parameter	Slope angle: 1:3	Slope angle: 1:2	Slope angle: 1:1.5
Tanα	0.333	0.5	0.6667
Depth at toe	3.59	3.59	3.59
H_S - m	2.26	2.26	2.26
T_P - s	13.3	13.3	13.3
ξ_{cr}	2.41	3.38	4.30
$\xi_{s-1,0}$	3.35	5.03	6.71
$H_S/H_{2\%}$	0.8	0.8	0.8
S_d	2	2	2
N – 2-hour storm	595.5	595.5	595.5
Permeable – P	0.4	0.4	0.4
Required Armour stone Dn_{50} - m	0.66	0.69	0.71
Required Armour stone Mn_{50} - kg	769.13	868.61	946.91
Impermeable – P	0.1	0.1	0.1
Required Armour stone Dn_{50} - m	0.79	0.93	1.05
Required Armour stone Mn_{50} - kg	1330.54	2164.40	3056.79

Table E-12: Armour stone requirement for the 100-year incident wave and year 2100 SWL

Parameter	Slope angle: 1:3	Slope angle: 1:2	Slope angle: 1:1.5
Tanα	0.333	0.5	0.6667
Depth at toe	3.63	3.63	3.63
H_S - m	2.30	2.30	2.30
T_P - s	13.6	13.6	13.6
ξ_{cr}	2.41	3.38	4.30
$\xi_{s-1,0}$	3.39	5.09	6.78
$H_S/H_{2\%}$	0.8	0.8	0.8
S_d	2	2	2
N – 2-hour storm	582.4	582.4	582.4
Permeable – P	0.4	0.4	0.4
Required Armour stone Dn_{50} - m	0.67	0.70	0.72
Required Armour stone Mn_{50} - kg	802.94	906.80	988.54
Impermeable – P	0.1	0.1	0.1
Required Armour stone Dn_{50} - m	0.81	0.95	1.07
Required Armour stone Mn_{50} - kg	1403.72	2283.45	3224.91

E.1.2 Addition of a face slope berm

To ensure a berm reduction factor, γ_b, of 0.6, the berm must conform to certain conditions. The method of calculating the berm reduction factor was reversed to obtain the desired berm length. According to CIRIA (2007), the berm functions best if located at the SWL.

The optimal berm length is calculated by finding the equivalent slope of the structure, α_{eq}. The slope, α_{eq}, is the gradient between two points, located on the structure, one incident wave height above and below the berm (See Figure E-1). When the equivalent slope is known, the required berm length can be calculated using simple geometric relationships. The method of calculating the berm reduction factor, γ_b, is shown in Equation E-3. This method was reversed, by using the optimal value for the berm reduction factor of 0.6 as input, to calculate the equivalent slope.

$$\gamma_b = 1 - r_B(1 - r_{dB}) \qquad \textbf{E-1}$$

Where,

γ_b = berm reduction factor, chosen as 0.6 (-)

r_B = $1 - \frac{\tan \alpha_{eq}}{\tan \alpha}$, and tanα is the average slope angle, 0.333 in this case.

r_{dB} = $0.5\left(\frac{d_B}{H_S}\right)^2$, and d_B = berm depth relative to the SWL, thus equal to 0 in this case.

The variable, r_{dB}, is reduced to 0 by placing the berm at the SWL line. Then Equation E-3 is reduced to:

$$\gamma_b = 1 - \left(1 - \frac{\tan \alpha_{eq}}{\tan \alpha}\right)$$

From where the equivalent slope, α_{eq}, can be found. Once the equivalent slope is found, the horizontal distance between the two points on the structure (1^*H_S above and below the SWL) can be calculated. The berm length is then calculated by subtracting the horizontal distances of the two points up and down the structure slope, from the total distance between the points.

See Table E-13 to Table E-15 for the calculation of the optimal berm lengths for the three slopes under consideration and for the various design conditions and lifetimes relevant in this study. The steepest slope, 1:1.5 requires the smallest berm and the most gradual slope 1:3 requires the largest berm structure.

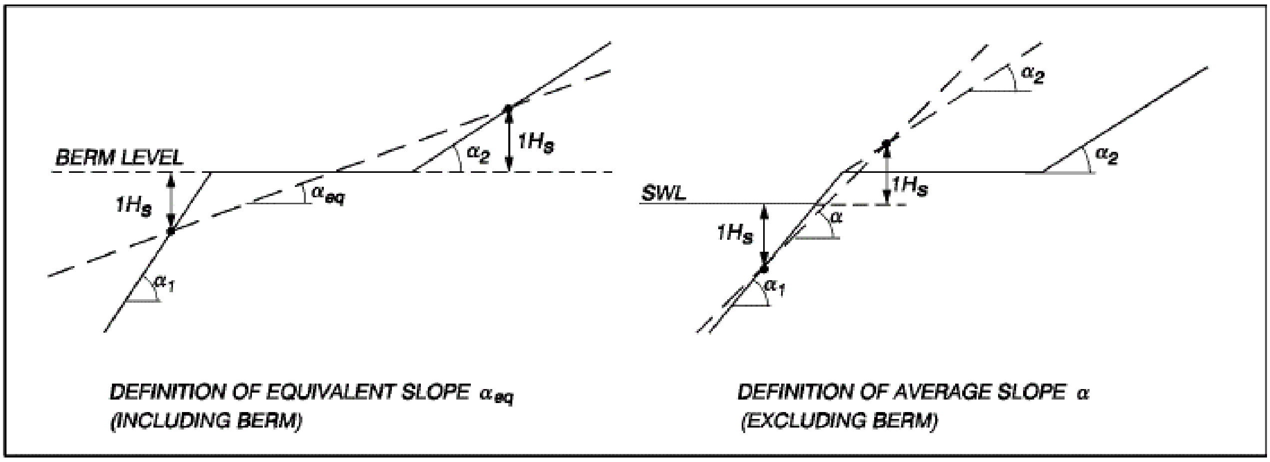


Figure E-1: Definition sketch of relative geometry features regarding the berm length calculation

Table E-13: Calculation of optimal berm length for a structure slope 1:3 and various design conditions

Parameter	Year 2030				Year 2050				Year 2100			
Design storm	**1:1**	**1:25**	**1:50**	**1:100**	**1:1**	**1:25**	**1:50**	**1:100**	**1:1**	**1:25**	**1:50**	**1:100**
α_{eq} (degrees)	11.3	11.3	11.3	11.3	11.3	11.3	11.3	11.3	11.3	11.3	11.3	11.3
Berm Length – b (m)	6.14	7.30	7.48	7.68	6.51	7.67	7.85	8.04	7.71	8.87	9.05	9.24

Table E-14: Calculation of optimal berm length for a structure slope 1:2 and various design conditions

Parameter	Year 2030				Year 2050				Year 2100			
Design storm	**1:1**	**1:25**	**1:50**	**1:100**	**1:1**	**1:25**	**1:50**	**1:100**	**1:1**	**1:25**	**1:50**	**1:100**
α_{eq}(degrees)	16.7	16.7	16.7	16.7	16.7	16.7	16.7	16.7	16.7	16.7	16.7	16.7
Berm Length – b (m)	4.09	4.86	4.98	5.11	4.33	5.10	5.22	5.35	5.12	5.90	6.02	6.15

Table E-15: Calculation of optimal berm length for a structure slope 1:1.5 and various design conditions

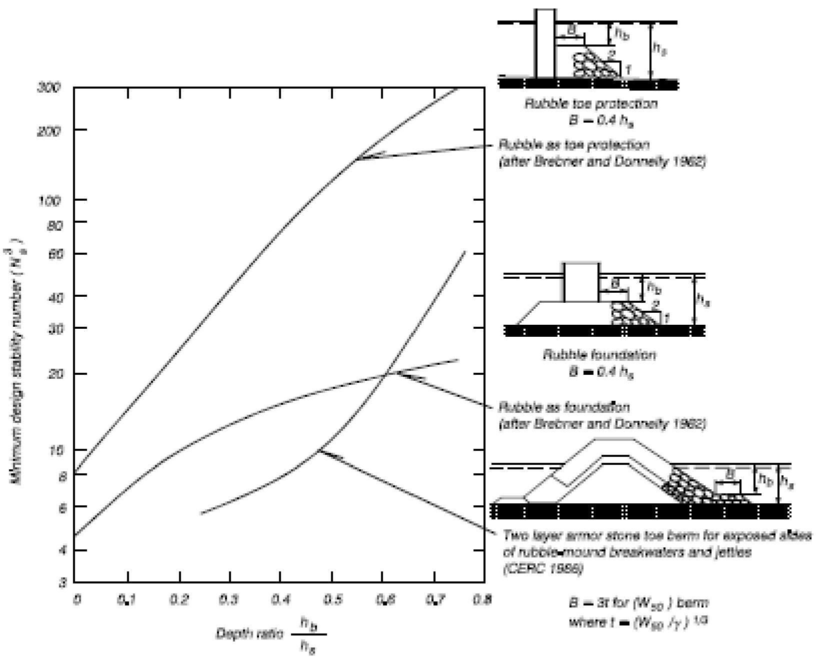
Parameter	Year 2030				Year 2050				Year 2100			
Design storm	**1:1**	**1:25**	**1:50**	**1:100**	**1:1**	**1:25**	**1:50**	**1:100**	**1:1**	**1:25**	**1:50**	**1:100**
α_{eq}(degrees)	21.8	21.8	21.8	21.8	21.8	21.8	21.8	21.8	21.8	21.8	21.8	21.8
Berm Length – b (m)	3.06	3.64	3.73	3.83	3.25	3.83	3.92	4.01	3.84	4.42	4.51	4.61

E.1.3 Toe stability

The method followed to ensure stability of the revetment toe armour rock is described in Figure E-2. The minimum stability curve for a two layer armour stone breakwater is used in this study as the minimum design stability criteria.

Stability of Toe Berm Tested in Regular Waves (Markle 1989)

Regular waves, head-on and oblique



where	N_s	$N_s = H/(\Delta D_{n50})$
	H	Wave height in front of breakwater
	Δ	$(\rho_s/\rho_w) - 1$
	ρ_s	Mass density of stones
	ρ_w	Mass density of water
	D_{n50}	Equivalent cube length of median stone

Remarks: The curves in the figure are the lower boundary of N_s-values associated with acceptable toe berm stability (i.e., some stone movement occurs; but the amount of movement is minor and acceptable, which shows that the toe is not overdesigned)

Figure E-2: Toe berm stability formulae (Source: USACE 2006)

E.2 Design of the Armoured Dike

The dike structure proposed for protection against achievable high water levels in the lower estuary basin, must be protected from scour induced by fast flowing water. Gabion mattresses were chosen for this purpose as retained rock reportedly performs better under hydraulic loadings than loose stone. For example, under the same hydraulic conditions, the stone required to fill the gabion mattresses can be up to three times as small as conventional armour rock for river bank protection (CIRIA 2007).

Table E-16: Typical values for hydraulic loads (Source: CIRIA 2007)

Situation	Return (U_r) or natural current	Water level depression		Secondary waves		Wind waves	
	Velocity (m/s)	Height Δh (m)	Period T (s)	Height H_i (m)	Period T (s)	Height H (m)	Period T (s)
Small river and restricted navigable channel	1.0–2.0 *	0.5–0.75	20–60	0.5	2.5	0.5	2
Large navigable channel	2.0	1.0	20–60	1.0	2.5	1.0	3–4
Large river and estuary	3.0–4.0	1.0	20–60	1.0	2.5	1.5–2.0	5–6

Due to the inability to calculate flow velocities for extreme conditions in the estuary basin, typical values for flow velocities in estuaries will be assumed. provides some typical values for various situations.

E.2.1 Gabion Mattresses

Table E-17 contains information of critical and limiting flow velocities for gabion mattress sizes and stone size.

Table E-17: Indicative values of critical and limiting velocities for gabion mattresses (CIRIA 2007)

Mattress Thickness (m)	Stone Size D_{n50} (mm)	Critical Velocity (m/s)	Limiting Velocity (m/s)
0.15 – 0.17	85	3.5	4.2
	110	4.2	4.5
0.23 – 0.25	85	3.6	5.5
	120	4.5	6.1
0.30	100	4.2	5.5
	125	5.0	6.4
0.5	150	5.8	7.6
	190	6.4	8.0

E.2.2 Impermeable core and drainage

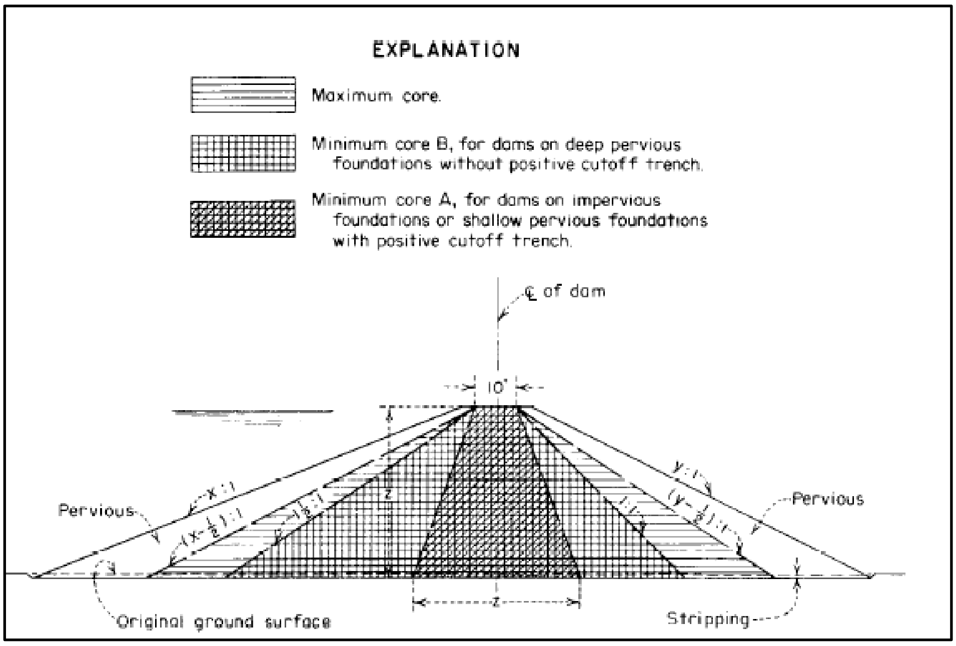


Figure E-3: Zoning design guidelines for embankment dams on pervious and impervious foundations (Source: USBR 1987)

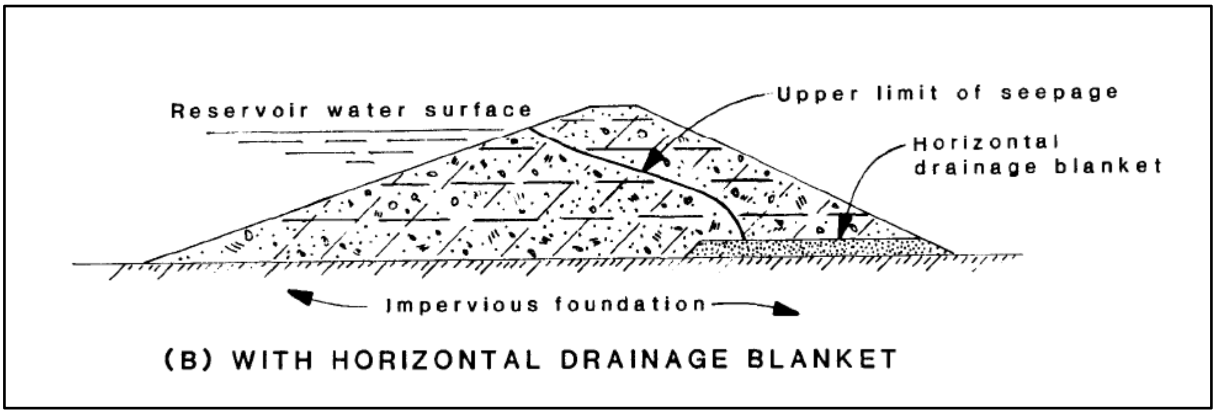


Figure E-4: Example of a horizontal drainage blanket in an embankment dam (Source: USBR 1987)

E.3 Order of magnitude cost estimate

The preferred defence option of a combined rock revetment and armoured dike structure will be subjected to an order of cost estimate. Construction costs were obtained from Roux (2017) and will subsequently be used to derive a cost estimate for the flood defence measure at the Great Brak estuary.

Table E-18: Relevant construction costs (Source: (Roux. 2017 pers. comm.)

Parameter	Value	Unit
Revetment costs		
Rock armour supply	750	R/m³
Rock under layer	650	R/m³
Rock placing	150	R/m³
Excavation in soft material	125	R/m³
Backfill	75	R/m³
Rolled clay	120	R/m³
Geotextile (supply and place)	50	R/m³
Concrete (mass)	2000	R/m³
Concrete (reinforced)	3500	R/m³
Reinforcing (supply and place)	20 000	R/t
Gabions		
Rock fill (supply)	700	R/m³
Rock fill (place)	150	R/m³
Zinc Gabion 2x1x1	685.14	R
Zinc PVC 2x1x1	1 063.62	R
Zinc mattress 6x2x0.3	1 686.06	R
Zinc PVC mattress 6x2x0.3	2 697.24	R
Additional		
Design fees	12	%
Ground surveys	5	%
EIA	300 000	R